

**INTERFACE SHEAR CAPACITY OF FACING UNITS OF
GEOSYNTHETIC-REINFORCED SEGMENTAL RETAINING
WALLS**

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ABSTRACT

The use of segmental retaining walls (SRWs) is in a period of development at the present time. Today, various types of segmental blocks are extensively used in many geotechnical applications in Malaysia and those blocks are imported from abroad or locally produced under licensed with the agreement of the foreign patent owners.

A specially designed and fabricated direct shear apparatus was developed at University of Malaya for full scale laboratory investigation of the innovated block. The developed apparatus was modified by considering the effects of fixed vertical piston on interface shear tests.

The experimental works were comprised of three groups of tests. Group 1 was divided into 3 configurations of tests series. The main variable among the test series was stiffness of shear pins. Stiffness of the shear pins varied from zero (no shear pins which allow block to move freely) to very high (steel pins). Another configuration was selected for a medium stiffness of shear pins (plastic pins) falling between the limiting stiffness cases (zero to very high). Frictional performance of hollow I-Block system was examined under three different normal load conditions.

Group 2 basically outlined the performance testing of the I-Block system infilled with granular in-fills. As granular in-fills, two types of recycled aggregates were selected and used along with natural aggregates. Recycled aggregates were mainly selected based on the compressive strength of the source waste concretes to investigate the effect of strength property on frictional behavior of recycled aggregates used as in-fillers.

Purely frictional capacity of I-Block infilled with recycled aggregates was compared to against those with infilled by fresh aggregates.

The tests of Group 3 were configured depending on the flexibility geosynthetic inclusions and granular in-fills. The primary objective of this group was to determine the performance parameters of the new block system with interlocking materials and geosynthetic inclusions. This group represents the potential field conditions of reinforced I-Block walls with proposed interlocking materials. In this group, three types of geosynthetic reinforcements were chosen: a flexible PET-geogrid, a stiff HDPE-geogrid, and a flexible PET-geotextile which are mostly used in Malaysia for GR-SRW constructions.

The results of the investigation report that interface shear capacity of the innovated block system greatly was influenced by interlocking mechanisms and interface stiffness. For example, the presence of shear connectors influenced the interface shear capacity depending on the nature of the connectors i.e. rigid or flexible. For the case of granular in-fills, it was found that granular infill definitely increases the interface shear capacity of the blocks compared to empty conditions. The frictional performance of blocks infilled with recycled aggregates is almost equal those with natural aggregates. The results showed that compressive strength of the source waste concretes has a little or no effect on the frictional performance of recycled concrete aggregates used into facing units. Inclusion of a geosynthetic layer at the interface had great influence on interface frictional performance of segmental retaining wall units. It depends on the flexibility of geosynthetic reinforcements as well as block's interlocking system. The evaluated results report that the angle of friction is greatly influenced by the inclusion's characteristics i.e. flexibility or rigidity than aggregate types.

ABSTRAK

Penggunaan dinding penahan bersegmen (SRWs) di Malaysia terutamanya di dalam aplikasi geoteknik semakin mendapat tempat dan sentiasa diperbaharui teknologinya dari semasa ke semasa melalui kajian yang dijalankan di peringkat universiti. Kebanyakan SRWs ini dihasilkan di dalam negara dan tidak kurang juga yang diimport dari luar. Samaada dihasilkan di dalam atau luar negara, SRWs ini mestilah mendapat kebenaran daripada pemilik paten terlebih dahulu.

Sebuah mesin ujian ricih untuk SRWs telah direkabentuk di Universiti Malaya bertujuan untuk mengkaji sifat dinding penahan bersegmen ini. Mesin ini telah diubahsuai dengan mengambil kira pelbagai faktor terutamanya dari segi kesan piston tegak yang tetap terhadap komponen ujian ricih. Spesimen dinding penahan bersegmen yang digunakan adalah sistem I-blok berongga.

Ujian eksperimen terbahagi kepada 3 jenis kumpulan. Kumpulan pertama terbahagi kepada 3 konfigurasi yang berlainan. Pengubah utama di dalam ujian adalah kekukuhan pin ricih. Kekukuhan pin ricih diukur daripada ujian yang tidak mempunyai pin dimana spesimen bergerak bebas (rendah) hingga ujian pin yang menggunakan pin besi (tinggi). Konfigurasi yang lain adalah penggunaan pin plastik (sederhana) yang terletak diantara julat rendah dan tinggi. Prestasi geseran sistem I-blok berongga dikaji dibawah 3 jenis keadaan beban normal.

Kumpulan 2 pula mengkaji prestasi sistem I-blok yang diisi dengan batuan granul (agregat). Agregat yang digunakan di dalam kajian terbahagi kepada 2 jenis, iaitu agregat kitar semula yang dipilih dan dicampurkan bersama agregat semulajadi. Agregat kitar semula dipilih berdasarkan kekuatan mampatan daripada bahan buangan

konkrit. Tujuannya adalah untuk mengkaji kesan sifat kekuatan ke atas sifat geseran agregat kitar semula yang digunakan sebagai bahan pengisi. Kapasiti I-blok yang diisi dengan agregat kitar semula dibandingkan dengan agregat semulajadi sebagai pengisi.

Kumpulan 3 pula mengkaji sifat fleksibel bahan geosintetik terhadap bahan pengisi iaitu agregat. Objektif utama kumpulan ini adalah untuk menentukan prestasi parameter sistem I-blok berongga yang digunakan bersama bahan pengikat dan bahan geosintetik. Kumpulan ini menggambarkan potensi keadaan dinding I-blok dengan bahan pengikat. Terdapat 3 jenis bahan geosintetik yang digunakan di dalam kumpulan ini iaitu PET-geogrid, HDPE-geogrid dan PET-geotekstil. Bahan geosintetik ini merupakan bahan yang digunakan secara meluas di Malaysia sebagai dinding penahan bersegmen.

Keputusan menunjukkan komponen kapasiti ricih sistem I-blok ini dipengaruhi oleh mekanisme pengikat dan komponen kekukuhan. Sebagai contoh, kehadiran pengikat ricih mempengaruhi komponen kapasiti ricih bergantung kepada sifat semulajadi bahan pengikat; tegar dan fleksibel. Untuk kes agregat sebagai pengisi, kajian mendapati agregat meningkatkan komponen kapasiti ricih I-blok dibandingkan dengan I-blok yang kosong. Prestasi geseran blok yang diisi agregat kitar semula adalah hampir sama dengan agregat semulajadi. Keputusan menunjukkan kekuatan mampatan konkrit buangan tidak mempengaruhi prestasi geseran agregat kitar semula yang diguna sebagai unit muka. Lapisan geosintetik pada blok pada komponen ricih pula mempengaruhi prestasi geseran pada unit dinding penahan bersegmen. Ia bergantung kepada prestasi fleksibiliti bahan geosintetik dan juga bahan pengikat dalam I-blok. Kajian mendapati sudut geseran dipengaruhi oleh sifat bahan agregat yang digunakan; fleksibiliti atau sifat tegar berbanding jenis agregat.

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LIST OF SYMBOLS

α/α_u	Peak apparent cohesion
α'/α'_u	Service state apparent cohesion
A_i	Total area of the interface surface (m ²)
A_{td}	Distance between two ribs of extruded geogrid (mm)
B_w	Bond width of extruded geogrid (mm)
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
$E_{(n)}$	Elevation of geosynthetic layer n above base of wall (m)
$F_{g(n)}$	Force in geosynthetic reinforcement layer n (kN/m)
F_p	Ultimate (Peak) shearing load (kN)
F_{ss}	Measured shear load at 6 mm deformation (kN)
H	Wall height (m)
H_u	Segmental concrete unit height (mm)
K_a	Coefficient of active earth pressure
L_u	Segmental concrete unit height (mm)
N	Normal stress (kPa) at block interface
P_{nom}	Nominal distance between two bonds of extruded geogrid (mm)
P_q	Resultant of active earth pressure due to applied uniform surcharge (kN/m)
$P_{q(H)}$	Horizontal component of active earth pressure from applied uniform surcharge (kN/m)
P_s	Resultant of active earth pressure from soil self-weight (kN/m)
$P_{s(H)}$	Horizontal component of active earth pressure from soil self-weight (kN/m)

q_l	Uniform surcharge live load at top of wall (kPa)
q_d	Uniform surcharge dead load at top of wall (kPa)
S_w	Strand width of extruded geogrid (mm)
T_c	Short term tensile strength of geosynthetic (kN/m)
T_b	Bond thickness of extruded geogrid (mm)
T_r	Rib thickness of extruded geogrid (mm)
V	Shear stress (kPa)
V_p/V_u	Peak (ultimate) shear capacity (kPa)
V_{ss}/V'_u	Service state shear capacity (kPa)
W_u	segmental concrete unit width (mm)
Z	Depth from the ground surface (m)
β	Back fill slope angle against horizontal (degrees)
γ_i	Unit weight of backfill soil in moist condition (degrees)
δ	Shear displacement (mm)
δ_i	Angle of friction between wall to soil (degrees)
λ/λ_u	Peak (ultimate) angle of friction between segmental concrete units (degrees)
λ'/λ'_u	Service state angle of friction between segmental concrete units (degrees)
σ_a	Lateral active earth pressure (kPa)

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CD	Cross-machine Direction
FHWA	Federal Highway Administration
FM	Fineness Modulus
GRS	Geosynthetic Reinforced Soil
GRI	Geosynthetic Research Institute
GR-SRW	Geosynthetic Reinforced Segmental Retaining Wall
HDPE	High Density Polyethylene
LVDT	Linear Variable Displacement Transducer
NCA	Natural Coarse Aggregate
NCMA	National Concrete Masonry Association
MCB	Modular Concrete Block
MD	Machine Direction
MSE	Mechanically Stabilized Earth
PET	Polyester
POFA	Palm Oil Fuel Ash
PP	Polypropylene
RC	Reinforced Concrete
RCA	Recycled Coarse Aggregate
SCU	Segmental Concrete Unit
SRW	Segmental Retaining Wall
SRWU	Segmental Retaining Wall Unit
UHMWPE	Ultrahigh Molecular Weight Polyethylene

CHAPTER 1 INTRODUCTION

1.1 General

Segmental retaining walls (SRWs) are in a period of development. They are used as the facing for geosynthetics reinforced soil retaining wall structures because of their sound performance, aesthetics, and cost-effectiveness, expediency of construction, good seismic performance, and ability to tolerate large differential settlement without any distress (Yoo and Kim, 2008). In Malaysia, the use of dry-stacked column of segmental units as the facing for retaining wall constructions has been extensively practiced for more than 10 years (Lee, 2000a).

Currently, various types of mortar-less concrete block systems are being used in Malaysia for slope stabilities, road constructions, bridge abutments, and landscaping purposes. Those block systems are imported from abroad or locally produced under licensed with the agreement of the foreign patent owners.

By considering technical and economic aspects with available blocks systems in the markets, a new type of block system (I-Block) is designed and developed locally, and used in this research.

Facing stability is an important issue in the current design guidelines (Berg et al., 2009; NCMA, 2010) and has an effect on internal stability analysis (Bathurst and Simac, 1997). Huang et al. (2003) also reported that block-block shear strength and block-reinforcement connection strength sturdily influence seismic stability of Geosynthetic Reinforced Segmental Retaining Walls (GR-SRWs). Past research works (Bathurst & Simac, 1993; Buttry et al., 1993; Soong & Koerner, 1997; Collin, 2001; Huang et al., 2007) reported that facing instability basically occurs due to poor connection strength

and inadequate connection systems. Facing stability is mainly controlled by performance parameters (shear and connection strength).

These parameters are evaluated only by full scale laboratory or field tests of blocks system used in segmental retaining walls.

One of the mechanisms of facing instability that needs a special attention by the engineers is the interface shear failure, which happens due to inadequate connection systems.

By considering the effect of normal loading arrangement on interface shear tests (Bathurst et al., 2008); a specially designed and modified apparatus is developed to carry out full scale laboratory study (performance tests) for the innovated segmental block system.

In this study, the performance of the modified test facility was identified. A full scale laboratory study was also conducted using that test facility to evaluate the performance parameters for the innovated block system under different interlocking systems and inclusions.

1.2 Research objectives

The specific objectives of this study are as follows:

1. To design and develop a test apparatus for full scale laboratory study of segmental retaining wall (SRW) units.
2. To develop and test an effective shear connector for the I-Block system.
3. To evaluate the interface shear capacity of the I-Block system infilled with recycled concrete aggregates (RCA).
4. To compare the interface shear capacity of I-Block system with three different types of geosynthetics' layers placed at the interface and three types of granular in-fills used in the tests.

1.3 Scope of the study

The scope of the study presented in this thesis has been limited significantly to the two aspects. Firstly it is limited to the design and development a test facility for full scale laboratory study of segmental retaining wall units at University of Malaya. Secondly it deals with the investigation of interface shear testing of the newly designed and locally produced I-Block system under different types of interlocking systems and inclusions at segmental concrete interface. In this study, the following tasks were completed to attain the research goals:

1. Design and development of a modified apparatus

An apparatus was developed at University of Malay to carry out full scale laboratory study of segmental concrete wall units. The developed apparatus was modified by considering the effects of fixed vertical piston on interface shear tests.

The modified apparatus allows the normal loading assembly (vertical piston) to move horizontally with the top block without affecting the surcharge load over the period of shear testing.

2. Effect of rigidity of shear connector

To compare the effects of mechanical connectors on interface shear behavior of modular block units, two types of shear pins (steel & plastic) were selected. Steel pins are normally used in segmental wall system to help out facing alignment. By considering the rigidity of steel pin, relatively flexible plastic made of UHMWPE was applied in this investigation. The influence of rigidity of shear pins on interface shear capacity was compared against purely frictional behavior.

3. Effect of recycled coarse aggregate as in-fillers

As granular in-fills, two types of recycled aggregates were used along with natural aggregates. Recycled aggregates were mainly selected based on the compressive strength of the source waste concretes to investigate the effect of strength property on frictional behavior of recycled aggregates used as in-fillers. Purely frictional capacity of I-Block infilled with recycled aggregates was compared to against those with infilled by fresh aggregates.

4. Effect of geosynthetic inclusion

The main objective of this part of investigation was to examine the effect of geosynthetic inclusion on interface shear capacity and frictional performance of geosynthetic reinforcement with recycled aggregates used as granular in-fills. In this investigation, three types of geosynthetic reinforcements were chosen: a

flexible PET-geogrid, a stiff HDPE-geogrid, and a flexible PET-geotextile which are mostly used in Malaysia for GR-SRW constructions.

1.4 Thesis organization

The contents of the thesis are organized into 6 important chapters:

Chapter 1 focuses on brief introduction, objectives and scopes of the current research.

Chapter 2 describes the design and development of the geosynthetic reinforced segmental retaining walls, and a review of some previously studied works about facing stability.

Chapter 3 provides an overview of the materials used in the laboratory investigation.

Chapter 4 describes the test facility, test methodology, instrumentation and data acquisition systems.

Chapter 5 presents the test results under different interface conditions and compares the measured results.

Chapter 6 interprets and compares the test results under different interface conditions.

Chapter 7 summaries the conclusions of this thesis work and give recommendations for the future research.

CHAPTER 2 LITERATURE REVIEW

2.1 General

The following chapter focuses the historical background and development of modern reinforced earth technology. It describes the mechanically reinforced earth walls (MSEWs) and its components including segmental retaining wall units (facing units) and geosynthetic reinforcements. It also provides an overview of design methodology of geosynthetic reinforced segmental retaining wall outlined in National Concrete Masonry Association (NCMA) design manual. Finally, a number of previous works related to the objectives of the current research are discussed.

2.2 Historical background of reinforced earth structures

Reinforced soil technology is ancient. Primitive people used natural materials such as straw, tree branches, and plant material to reinforce the earth for centuries. The Ziggurats of Babylonia (Tower of Babel) were built by reinforcing soil with reed mats about 2,500 to 3,000 years ago in Mesopotamia (now Iraq). The Great Wall of China (2,000 BC) is another example of an ancient reinforced soil structure, where tamarisk branches were used to reinforce the portions of wall (Collin, 1997; Fitzpatrick, 2011).

The earliest version of an engineered reinforced soil wall called Mur Echelle (ladder wall), which was invented by Andre Coyne in 1929. A schematic of the ladder wall is shown in Figure 2.1. As first structure, a 4.5 m high quay-wall was constructed using this system in Brest, France in 1928. Unluckily, the application of Mur Echelle was discontinued after World War II (Lee, 2005).

The modern rediscovery of reinforced soil retaining wall system was pioneered by French architect and engineer Henri Vidal in the early 1960's (Barry, 1993; Carter and Dixon, 1995; Isabel et al., 1996; Berg et al., 2009). He invented new technique and modernized the reinforced soil retaining wall system. This system is called "Terre Armee" where horizontal metal strips are used with precast concrete facing panels to reinforce the backfill soil (Leblanc, 2002). This is also known as mechanically stabilized earth (MSE) system. The first wall was built using Vidal technology in United States in 1972 (Berg et al., 2009) and it has gained popularity throughout the world, mainly because of economical and aesthetics value.

In the 1970's, this MSE technology segued into polymeric reinforcement with the advent of geosynthetic materials (Lee, 2000b; Bourdeau et al., 2001; McGown, 2009). Geosynthetics have been used as an alternative (to steel) reinforcement material for reinforced soil structures due to its many fold advantages. The first geotextile-reinforced wall was found in France, which was built in 1971. After the development of geogrid polymers, it was firstly used in soil reinforcement in 1981. Since then, the application of geosynthetic reinforced soil (GRS) structures has increased rapidly (Berg et al., 2009; Hossain et al., 2009).

Now, a variety of facing systems are used in retaining wall constructions with modern geosynthetics. Among the facing systems of the mechanically stabilized earth (MSE) retaining walls, segmental retaining walls (SRWs) also called modular concrete block walls are in a period of enormous growth at the present time. The use of segmental concrete units as the facing for geosynthetic MSE walls has been frequently used since their first appearance in the mid 1980's (Bathurst and Simac, 1994).

Since 1990, the use of geosynthetic reinforced walls has increased dramatically by the introduction of segmental retaining wall (SRW) units (Hossain et al., 2009).

Nowadays, Geosynthetic Reinforced Segmental Retaining Walls (GR-SRWs) as earth structures are frequently used in many geotechnical applications due to their sound performance, aesthetically pleasing finishes, cost effectiveness, and ease of construction. In Malaysia, geotechnical engineers have been widely practicing GR-SRWs for the last decades (Lee, 2000a).

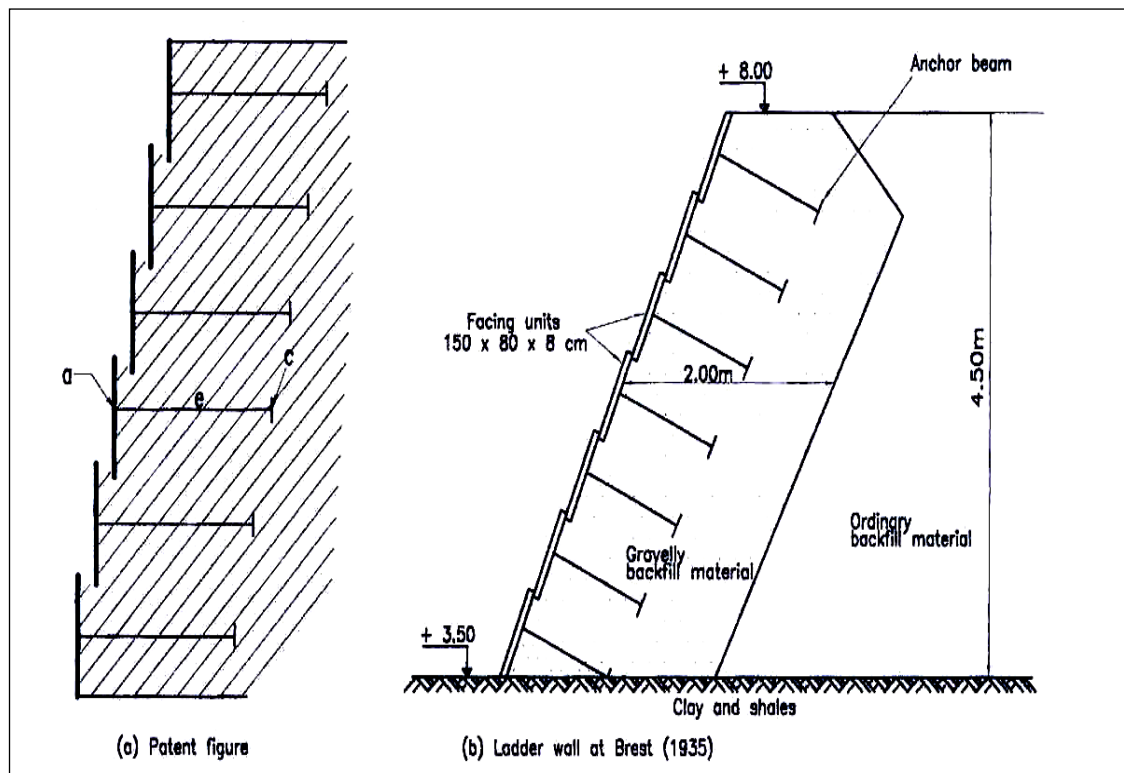


Figure 2.1: Schematic illustration of ladder wall (Lee, 2005)

2.3 Mechanically stabilized earth walls

According to The American Association of State Highway and Transportation Officials (AASHTO), Mechanically Stabilized Earth (MSE) walls are earth retaining structures (Figure 2.2) that employ either metallic (strip or grid type) or polymeric (sheet, strip or grid type) tensile reinforcements in a soil mass, and a facing element which is vertical or near-vertical (AASHTO, 1996). MSE walls performance as gravity walls that restrain lateral forces through the dead weight of the composite soil mass behind the facing column. The self-weight of the relatively thick facing may also contribute to the overall capacity. MSE walls are relatively flexible and often used where conventional gravity, cantilever or counter fort concrete retaining walls may be subject to foundation settlement due to poor subsoil conditions (Leblanc, 2002).

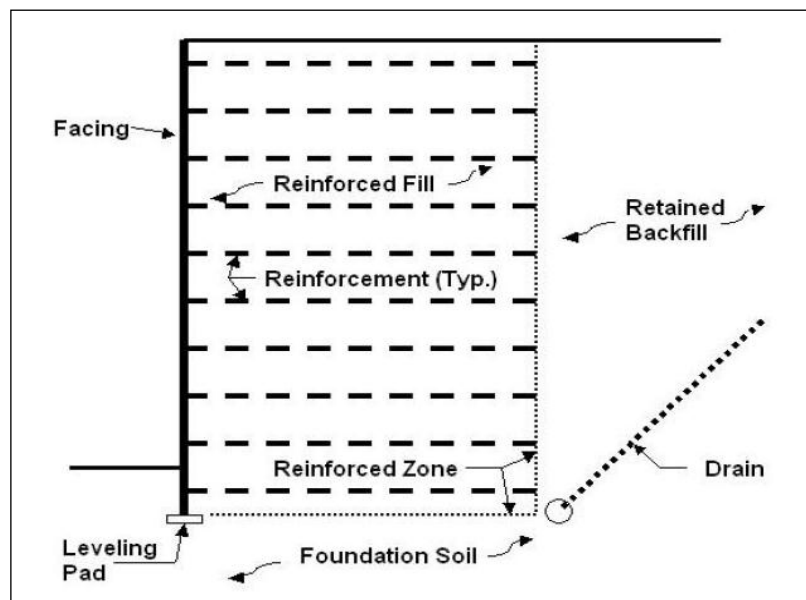


Figure 2.2: Cross section of a typical MSE structure (Berg et al., 2009)

Koerner and Soong (2001) grouped MSE walls into the following categories and subcategories depending on tensile reinforcements and facing elements:

1. MSE walls with metal reinforcement
 - a. Precast concrete facing panels
 - b. Cast-in-place facing
 - c. Modular block facings (Segmental retaining walls)
2. MSE walls with geosynthetic reinforcement
 - a. Wrap-around facing
 - b. Timber facing
 - c. Welded-wire mesh facing
 - d. Gabion facing
 - e. Precast full-height concrete facing
 - f. Cast-in-place full-height facing
 - g. Precast panel wall facing units
 - h. Segmental concrete walls (SRWs) (modular block facings)

Different types of facing systems for geosynthetic reinforced soil are illustrated in Figure 2.3.

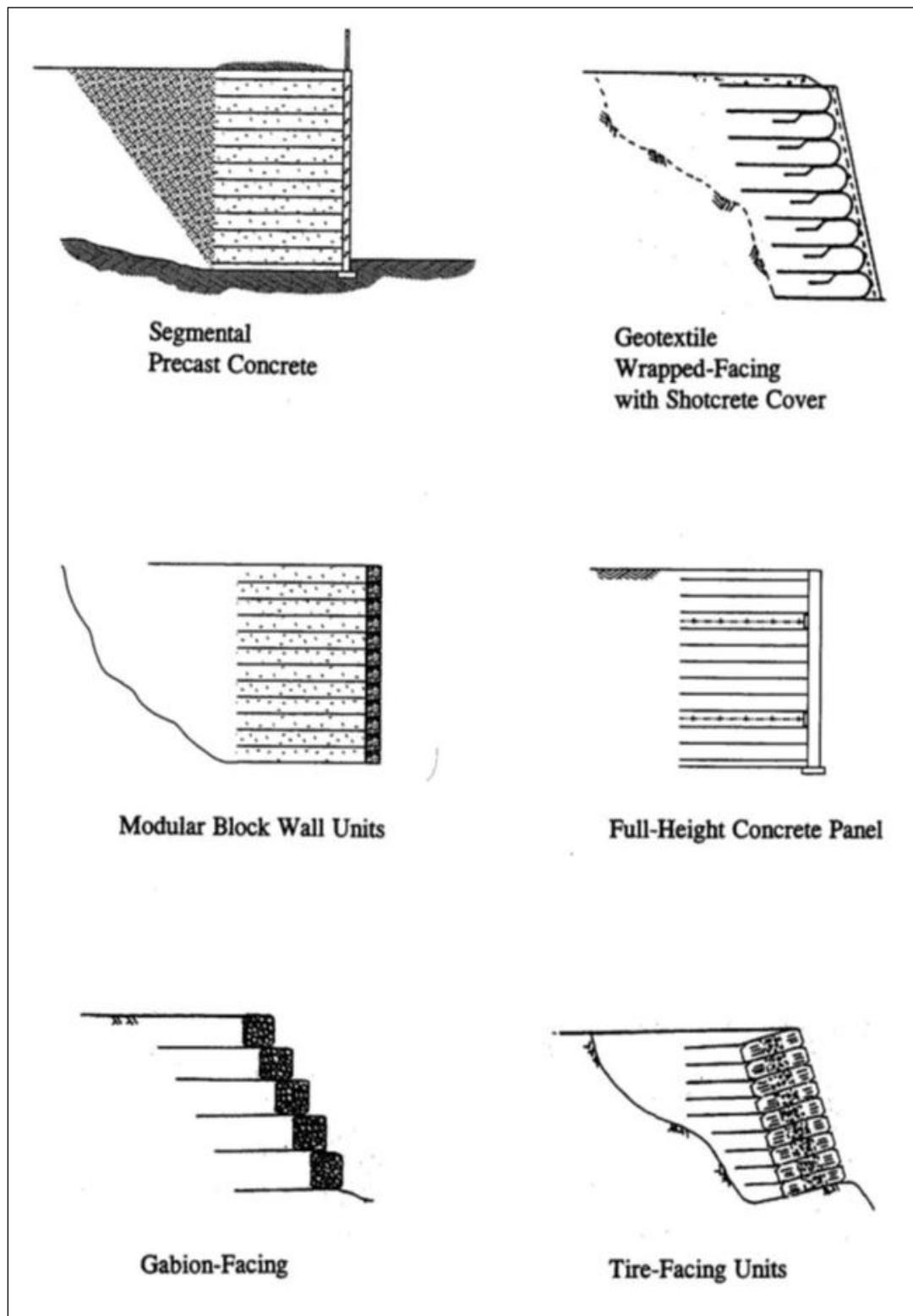


Figure 2.3: Facing types for geosynthetic reinforced soil wall (Berg et al., 2009)

MSE walls are cost-effective alternatives to conventional retaining walls. It has been noticed that MSE walls with precast concrete facings are usually less expensive than reinforced concrete (RC) retaining walls for heights greater than about 3 m (Berg et al., 2009). A cost survey of retaining walls was conducted by different individuals and agencies as shown in Table 2.1. According to Koerner and Soong (2001), Lee et al. (1973) subdivided the walls into high ($H \geq 9.0$ m), medium ($4.5 < H < 9.0$) and low ($H \leq 4.5$ m) height categories. Berg et al. (2009) also reported that the use of MSE wall results in a 25 to 50% cost saving than a gravity wall (Figure 2.4). The plots of the Figure 2.4 are drawn using the survey data, which was conducted by Koerner et al. (1998) under U.S. departments of Transportation. From the Figure 2.4, it is seen that gravity walls are most expensive over all wall categories with all wall heights. MSE walls with geosynthetic reinforcements are most cost-effective, although MSE (metal) walls significantly less expensive. Figure 2.4 also shows that crib walls are rare more than 7 m in height.

Table 2.1: Cost comparison of past retaining walls with wall height (units are U.S. dollars per square meter of wall facing) (Koerner et al., 1998)

Wall category	Wall height (relative)	Lee et al. (1973)	VSL Corporation (1981)	Yako and Christopher (1998)	GRI (1998)
Gravity Walls	High	300	570	570	760
	Medium	190	344	344	573
	Low	190	344	344	455
Crib/Bin Walls	High	245	377	377	I/D
	Medium	230	280	280	390
	Low	225	183	183	272
MSE (metal) Walls	High	140	300	300	385
	Medium	100	280	280	381
	Low	70	172	172	341
MSE (geosynthetic) Walls	High	N/A	N/A	250	357
	Medium	N/A	N/A	180	279
	Low	N/A	N/A	130	223

Notes: I/D = inadequate data; N/A = not available at that time

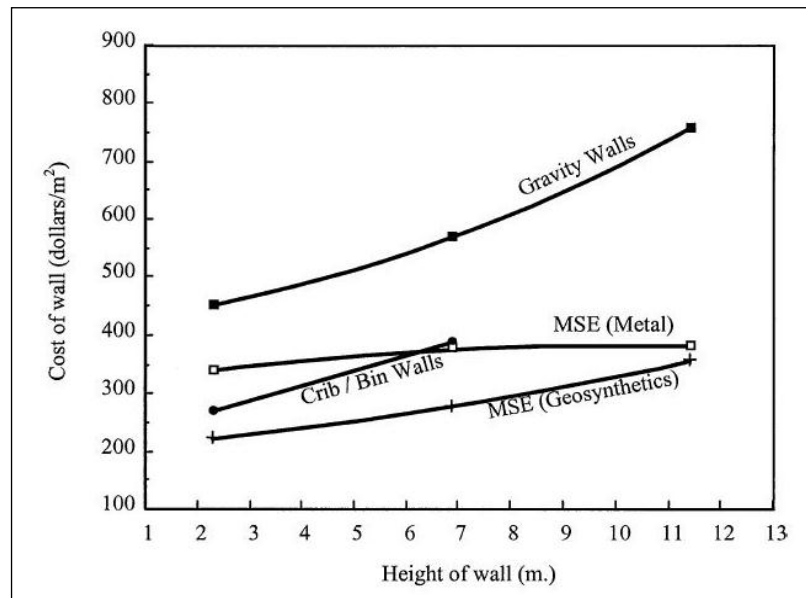


Figure 2.4: Cost comparison of retaining walls (Koerner et al., 1998)

2.4 Segmental retaining walls

A segmental retaining wall (SRW) is erected from dry-stacked units (mortar-less) that are usually connected through concrete shear keys or mechanical connectors. Segmental retaining walls are divided into two groups according to soil reinforcement: conventional SRWs and reinforced soil SRWs. Conventional SRWs are structures that resist external destabilizing forces, solely through the self-weight and batter of the facing units. Reinforced soil SRWs are composite systems consisting of mortar-less facing units in combination with a reinforced soil mass stabilized by horizontal layers of geosynthetic or metallic reinforcements. Figure 2.5 shows schematic diagrams of SRW systems and their components. Reinforced soil SRWs are also referred as MSE walls. SRWs offer important advantages over other types of soil retaining wall systems due to their durability, outstanding aesthetics, ability to tolerate differential settlement, ability to incorporate curves or corners, ease of installation and economics.

Segmental concrete walls (SRWs) also called modular concrete block (MCB) walls are in a period of enormous growth at the present time. They are frequently used in a number of applications including landscaping walls, structural walls, bridge abutments, stream channelization, waterfront structures, tunnel access walls, wing walls and parking area support (Collin, 1997). Figure 2.6 demonstrates the different applications of segmental retaining walls.

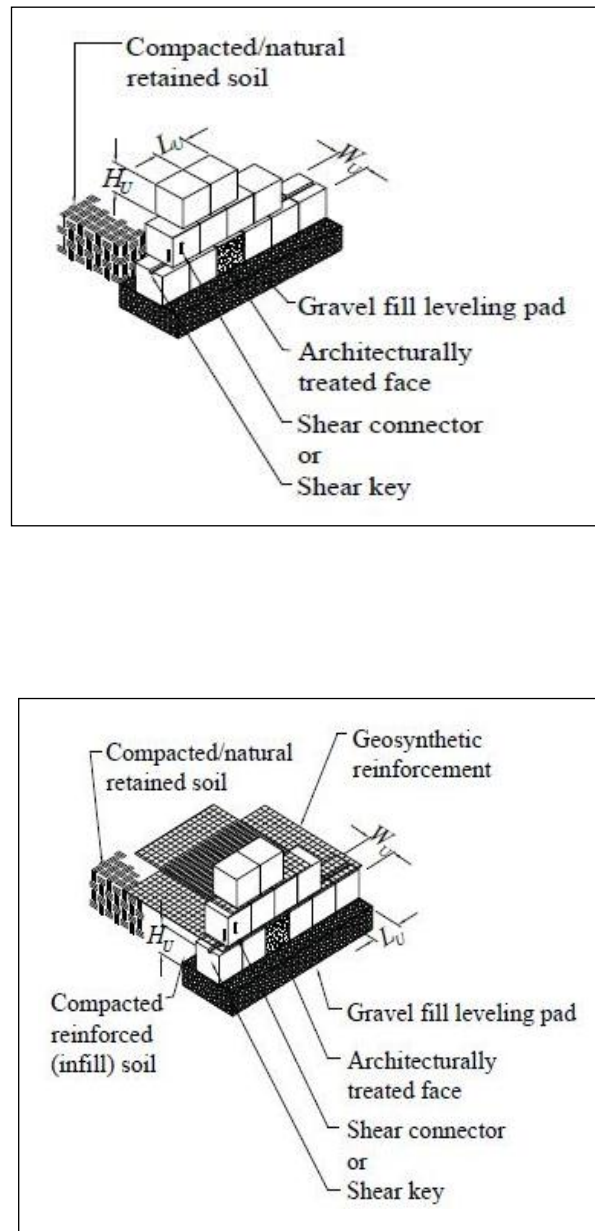


Figure 2.5: Segmental retaining wall systems (Collin, 1997); conventional (top) and Reinforced soil (bottom) SRW

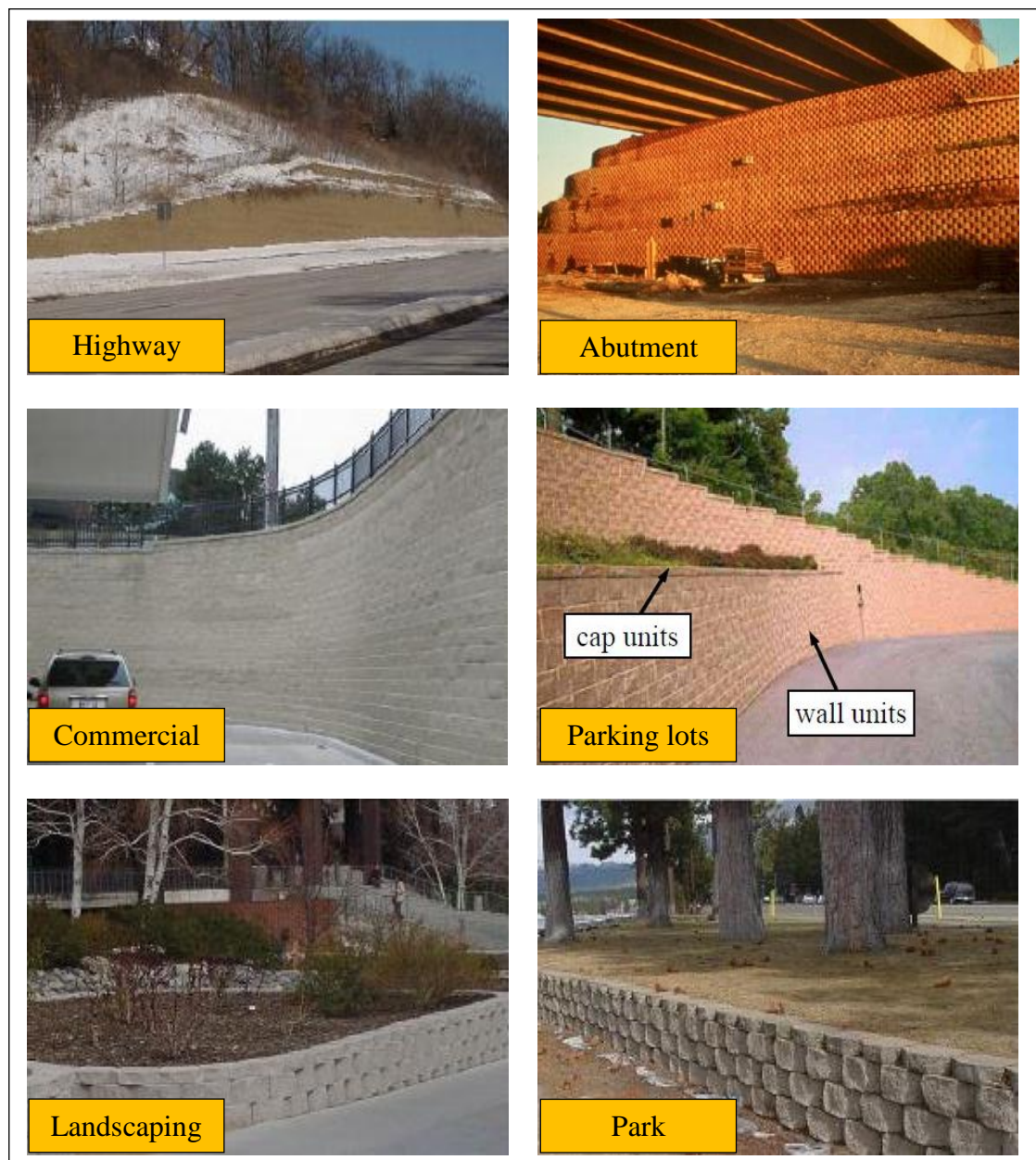


Figure 2.6: Applications of SRW systems (adapted from Chan et al., 2007; Chan et al., 2008; Bathurst, IGS)

2.5 Segmental retaining wall units

Segmental retaining wall (SRW) units are precast concrete units produced using wet or dry casting (machine molded) processes without internal reinforcement. The units may be manufactured solid or with cores, and the cores in and between the blocks are filled with aggregates during erection of wall. These units are also known as segmental concrete units (SCUs) or modular concrete blocks (MCBs). These precast units provide temporary formwork for reinforced soil SRWs during the placement and compaction of backfill soils. Figure 2.7 illustrates a variety of available proprietary segmental concrete units with different in size, shape, surface texture, and interlocking mechanism. The size, shape, and mass of a unit vary in wide range because there are no limitations on them. Most proprietary units are typically 80 to 600 mm in height (H_u), 150 to 800mm in width (W_u) (toe to heel) and 150 to 1800mm in length (L_u) (Bathurst and Simac, 1997). The mass of SRW units usually varies from 15 to 50 kg and the units of 35 to 50 kg normally are used for highway works (Berg et al., 2009). A variety of surface textures and features are available, including split faced, soft split faced, and stone faced, and molded face units, anyone of which may be scored, ribbed, or colored to fit any architectural application (TEK 2-4B, 2008).

Segmental concrete units are discrete units which are stacked in running bond configuration. To develop interlocking mechanism between successive vertical courses of these units, two different types of shear connections are mainly used in retaining wall constructions. One is built-in mechanical interlock in the form of concrete shear keys or leading/trailing lips and another one is the mechanical connector consisting of pins, clips, or wedges (Figure 2.8). Shear connections also maintain the horizontal setback in between successive segmental unit rows and also assist in controlling a constant wall

facing batter. Facing batter angles typically range from 1° to 15° . The connection systems also help to grip and align geosynthetic materials in place (Collin, 1997).

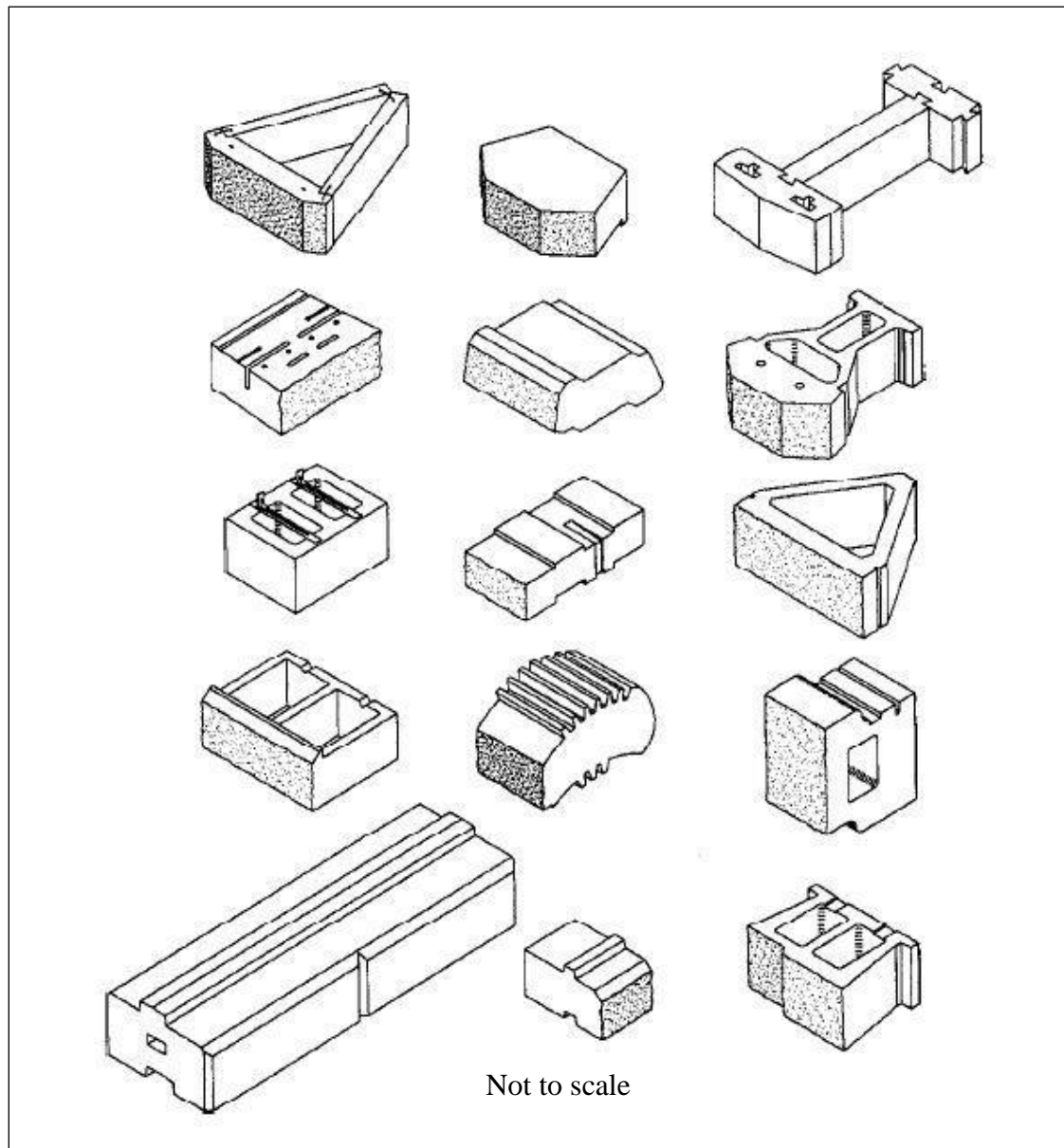


Figure 2.7: Examples of commercially available SRW units (Bathurst and Simac, 1997)

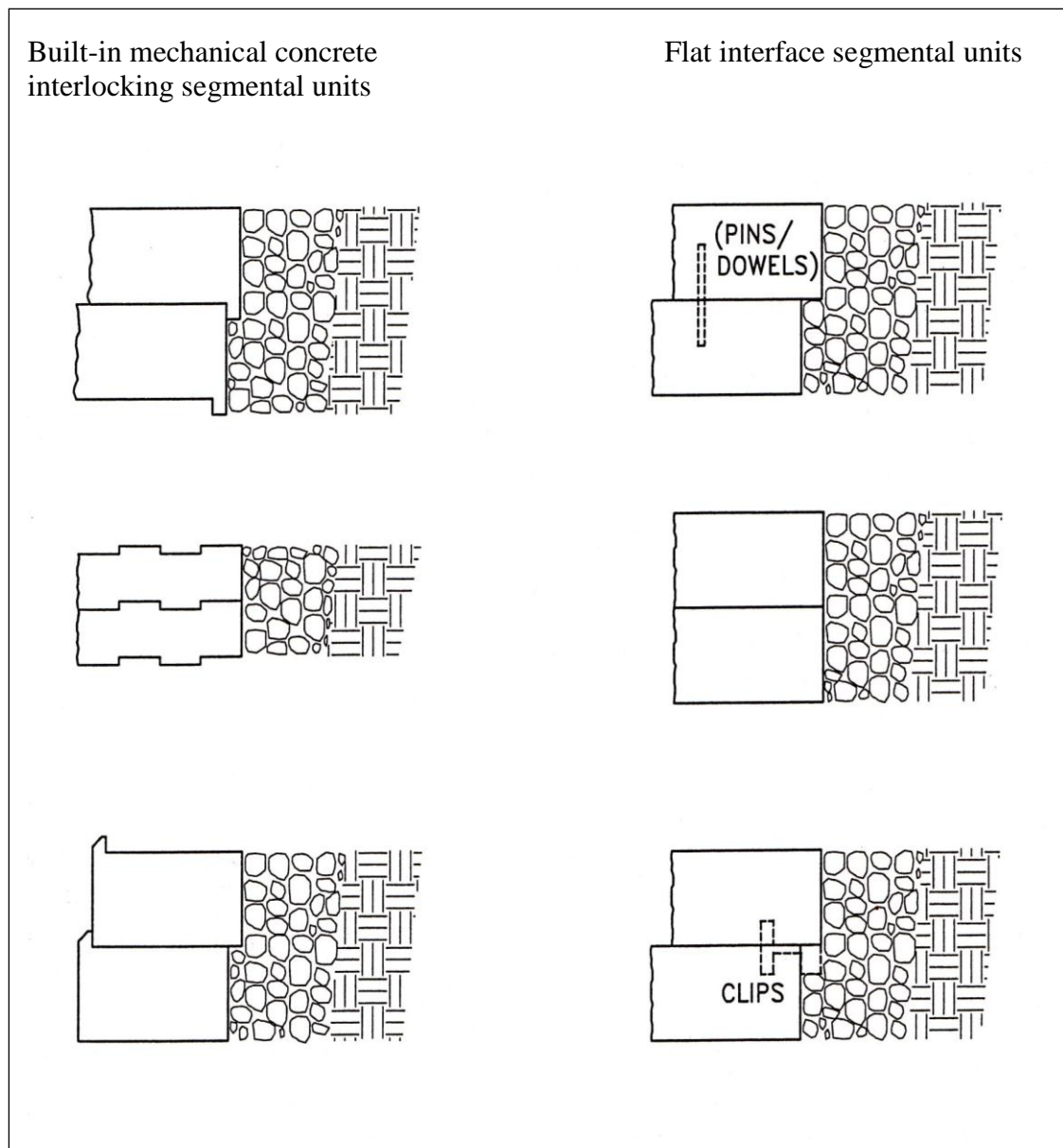


Figure 2.8: Shear connection types of SRW units (Collin, 1997)

2.6 Geosynthetic materials

Geosynthetics have been effectively used all over the world in different fields of civil engineering for the last four decades (Bourdeau et al., 2001; Shukla and Yin, 2006; Palmeira et al., 2008). Geosynthetics are now a well-accepted construction material and extensively practiced in many geotechnical, environmental and hydraulic engineering applications. In comparison with conventional construction materials, the use of geosynthetic offers excellent economic alternatives to the conventional solutions of many civil engineering problems. Geosynthetics have become essential components of modern soil stabilizing systems such as retaining walls or slopes (Shukla, 2002; Koseki, 2012). The use of geosynthetics in reinforced soil system has been accelerated by a number of factors such as; aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress (Zornberg, 2008). The use and sales of geosynthetic materials are frequently increasing at rates of 10% to 20% per year (Class Note, 2003).

Geosynthetics are planar products manufactured from polymeric materials (the *synthetic*) used with soil, rock, earth, or other geotechnical engineering (the *geo*) related material as an integral part of a man-made project, structure, or system (ASTM D 4439). Geosynthetics is a common term used to describe a broad range of polymeric products used in soil reinforcement and environmental protection works. Bathurst (2007) classified the geosynthetics into the following categories based on method of manufacture:

1. Geotextiles (GT)
2. Geogrids (GG)
3. Geonets (GN)
4. Geomembranes (GM)
5. Geocomposites (GC)
6. Geosynthetic Clay Liners (GCL)
7. Geopipes (GP)
8. Geocells (cellular confinement) (GL)
9. Geofoam (GF)

A convenient classification system for geosynthetics is illustrated in Figure 2.9 and the details can be found in Rankilior (1981), Koerner (1986) and Ingold and Miller (1988). Generally, Most of the geosynthetics are manufactured from synthetic polymers, which are materials of very high molecular weight, and highly resistant to biological and chemical degradation. Table 2.2 outlines the polymers used for producing geosynthetics along with their commonly used abbreviations. Among different types of polymers; polypropylene (PP), high density polyethylene (HDPE) and polyester (PET) are most commonly used in geosynthetic productions. The properties of some of the polymers listed in Table 2.2 are compared in Table 2.3. The typical strength-extension curves of these polymer types under short term load conditions are shown in Figure 2.10. Natural fibers (biodegradable) such as cotton, jute, coir, and wool are also used as raw materials for biodegradable geosynthetics (like geojute), which are mainly applied for temporary works (Shukla, 2002; Holtz, 2003; Shukla and Yin, 2006). Geosynthetics are commonly identified by polymer, type of fiber or yarn and manufacturing process.

Geosynthetics have very diverse application area in civil engineering. They are mainly defined by their primary or principal function (Table 2.4). In addition to the primary function, geosynthetics also perform one or more secondary functions in many applications. So it is important to consider both of the primary and secondary functions in the design considerations. Figure 2.11 demonstrates the six basic functions of geosynthetics.

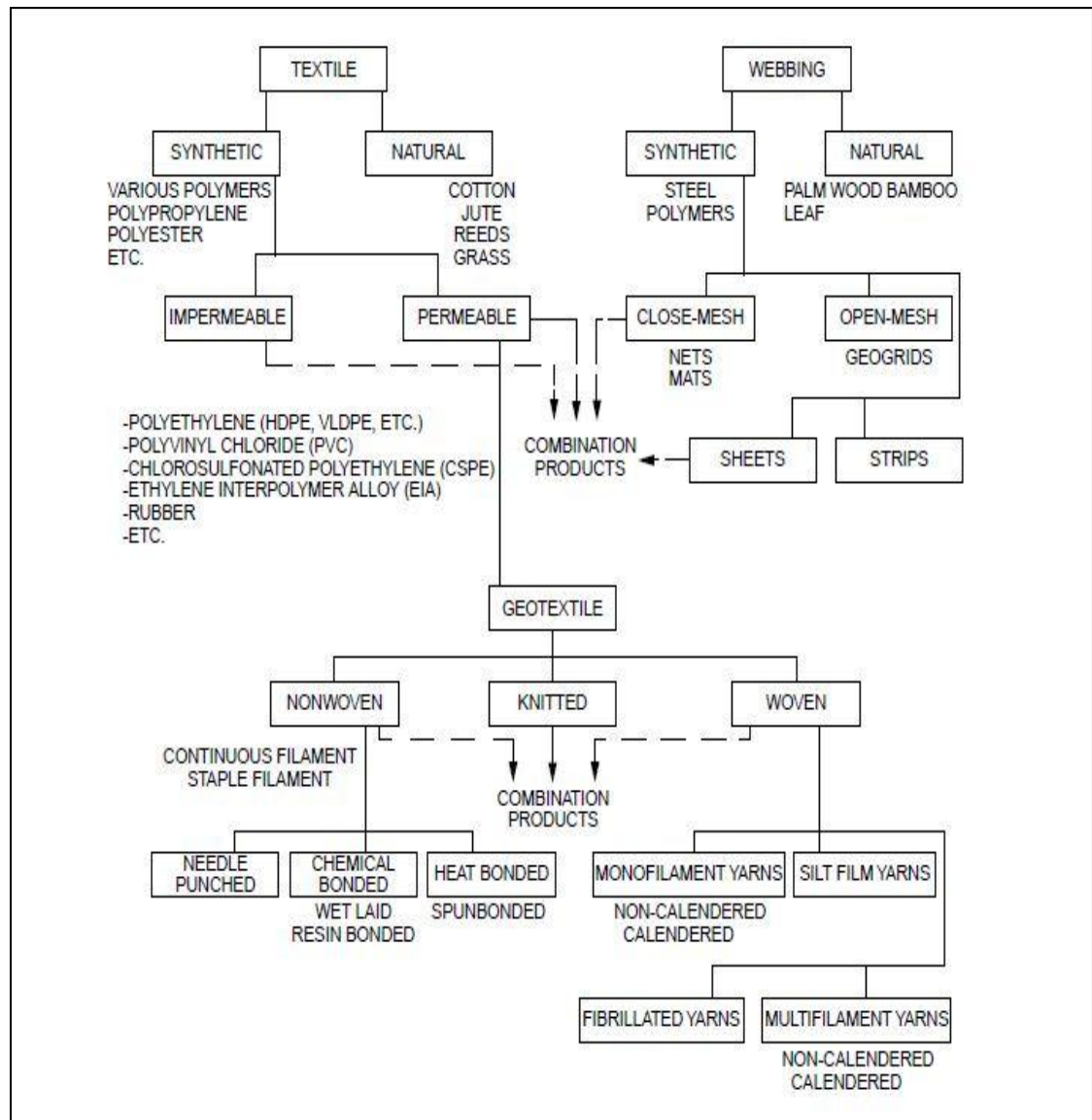


Figure 2.9: Classification of geosynthetics (Holtz, 2003)

Table 2.2: Polymers generally used for manufacturing geosynthetics
(Shukla and Yin, 2006)

Type of polymer	Abbreviations
Polypropylene	PP
Polyester (polyethylene terephthalate)	PET
Polyethylene	
Low density polyethylene	LDPE
Very low density polyethylene	VLDPE
Linear low density polyethylene	LLDPE
Medium density polyethylene	MDPE
High density polyethylene	HDPE
Chlorinated polyethylene	CPE
Chlorosulfonated polyethylene	CSPE
Polyvinyl chloride	PVC
Polyamide	PA
Polystyrene	PS

Table 2.3: A comparison of properties of polymers used in the production of geosynthetics (Shukla, 2002)

Property		Polymers			
		PP	PET	PA	PE
Strength		Low	High	Medium	Low
Modulus		Low	High	Medium	Low
Strain at failure		High	Medium	Medium	High
Creep		High	Low	Medium	High
Unit weight		Low	High	Medium	Low
Cost		Low	High	Medium	Low
Resistance to ultraviolet light	Stabilized	High	High	Medium	High
	Unstabilized	Medium	High	Medium	Low
Resistance to alkalis		High	Low	High	High
Resistance to fungus, vermin, insects		Medium	Medium	Medium	High
Resistance to fuel		Low	Medium	Medium	Low
Resistance to detergents		High	High	High	High

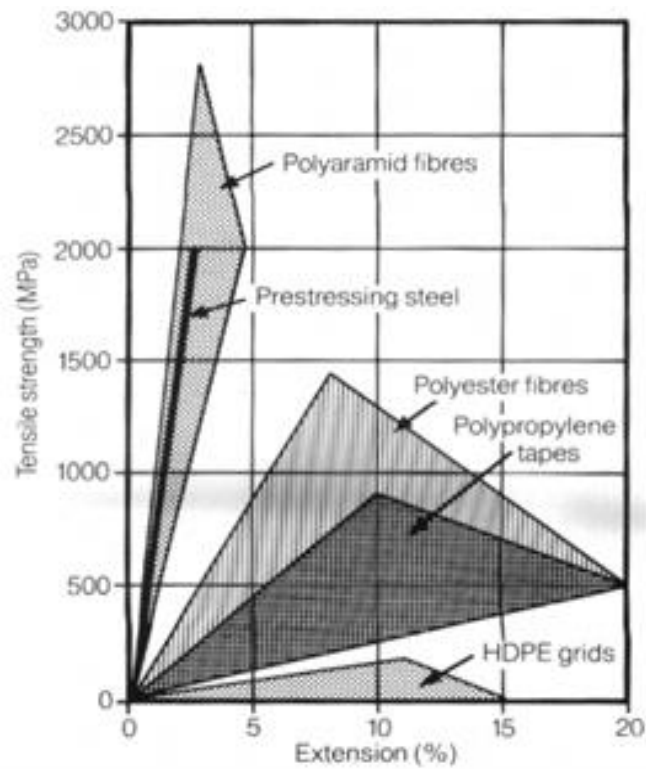


Figure 2.10: Typical strength behaviors of some polymers (Smith, 2001)

Table 2.4: Primary function of different geosynthetics
(adapted from Zornberg and Christopher, 2007)

Types	Separation	Reinforce- ment	Filtration	Drainage	Fluid Barrier	Protection
Geotextile	X	X	X	X	X ^a	X
Geogrid		X				
Geonet				X		
GM					X	
GCL					X	X
Geofoam	X					
Geocells		X		X		X
GC	X	X	X	X	X	X ^a

^aConditional geosynthetics

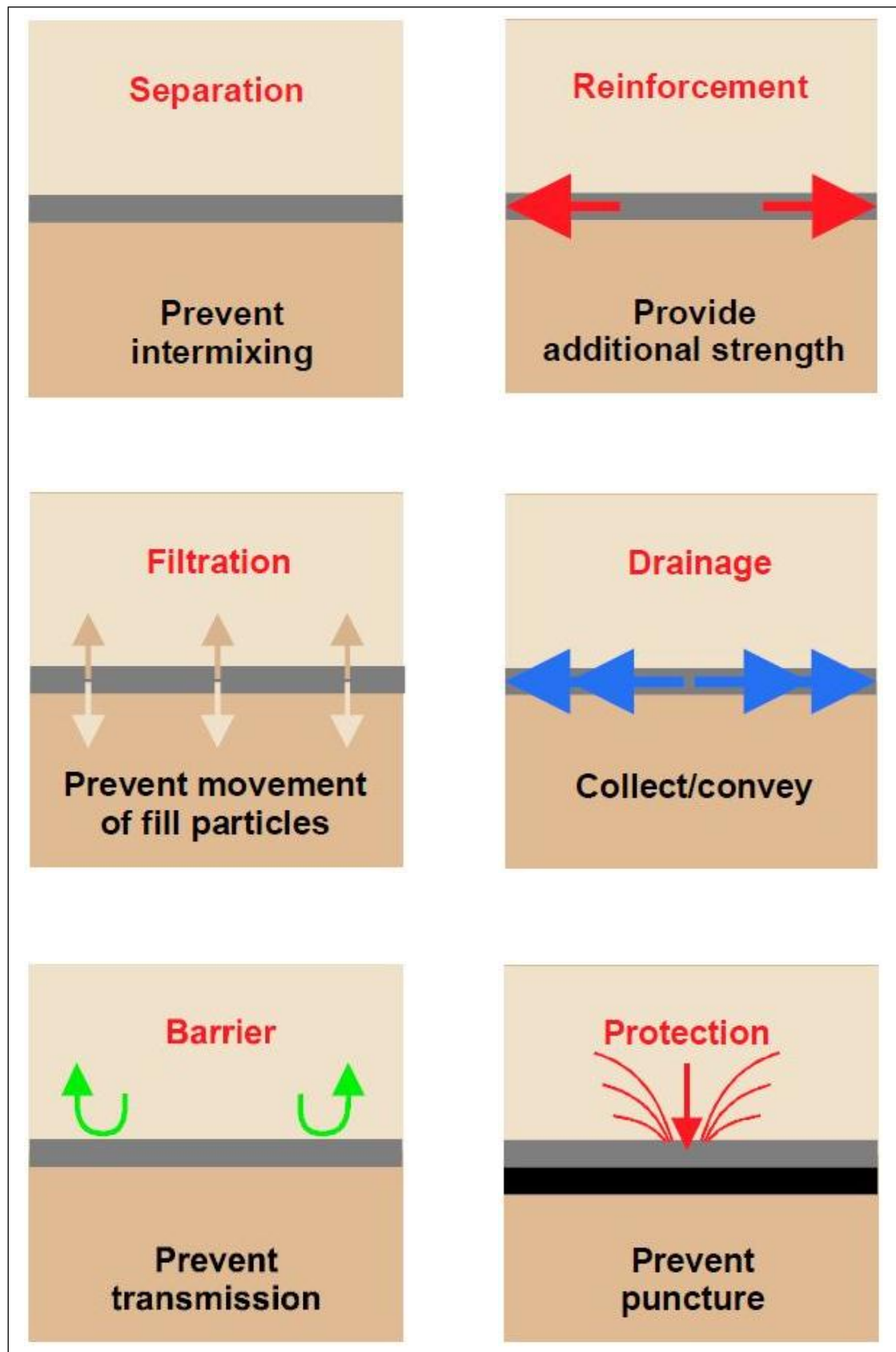


Figure 2.11: Basic functions of geosynthetics (Geofrabricks Ltd)

One of the most important functions of geosynthetics is soil reinforcement, where geosynthetics add tensile strength to a soil mass (Figure 2.12). Hence, a soil mass with geosynthetic inclusions acts as a composite material (reinforced soil), and possess high compressive and tensile strength (similar to the reinforced concrete). The three main applications of geosynthetics in soil reinforcement are (1) reinforcing the base of embankments constructed on very soft foundations, (2) increasing the stability and steepness of slopes, and (3) reducing the earth pressures behind retaining walls and abutments (Holtz, 2001). Geotextiles (woven and non-woven) and geogrids are typically used for soil reinforcement. So a brief literature will be focused on these specific families of geosynthetics.

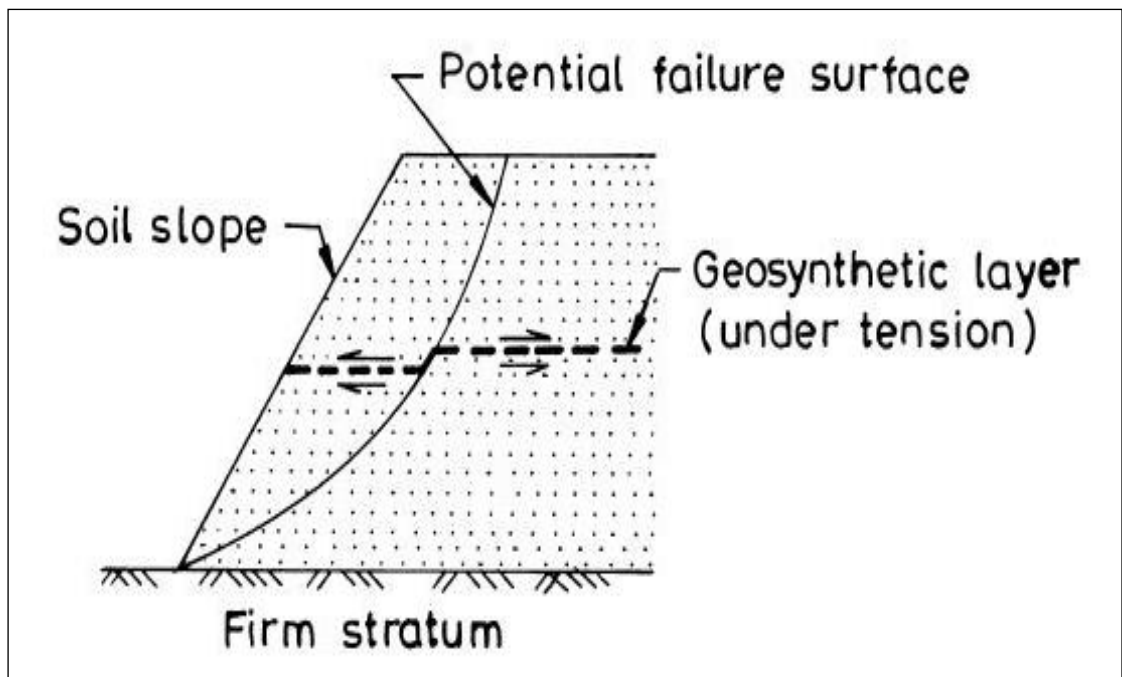


Figure 2.12: Basic mechanism of geosynthetic-soil composite (Shukla and Yin, 2006)

2.6.1 Geotextiles

ASTM (2003) has defined geotextiles as permeable geosynthetics made from textile materials. Geotextiles are one of the largest parts of geosynthetics and they have widest range of properties among different types of geosynthetic products. The primary functions of geotextiles are filtration, drainage, separation, and reinforcement. They also perform some other secondary functions listed in Table 2.4.

Geotextiles are manufactured from polymer fibers or filaments of polypropylene, polyester, polyethylene, polyamide (nylon), polyvinyl chloride, and fiberglass. In manufacturing of geotextiles, polypropylene and polyester are mostly used (Shukla, 2002; Basham et al., 2004). The most important reason of using polypropylene in geotextile manufacturing is its low cost, and high chemical and pH resistance (Table 2.3). Approximately 85% of the geotextiles used today are made from polypropylene resin. An additional 10% are polyester and the remaining 5% are made from other polymers (Zornberg and Christopher, 2007).

In manufacturing geotextiles, different types of fibers or filaments are used and the most common types are monofilament, multifilament, staple filament, and slit-film (Figure 2.13). Yarns are a bundle of fibers which are twisted together by spinning process. Monofilaments are produced by extruding the molten polymer through an apparatus containing small-diameter holes. The extruded polymer strings are then cooled and stretched to give the filament increased strength. Staple filaments are also made by extruding the molten polymer and then extruded filaments are cut into 25 to 100 mm portions. The staple filaments are spun to form longer staple yarns. Slit-film filaments are created by either extruding or blowing a film of a continuous sheet of polymer and cutting it into filaments by knives or lanced air jets.

Slit-film filaments have a flat, rectangular cross-section instead of the circular cross-section shown by the monofilament and staple filaments (Zornberg and Christopher, 2007).

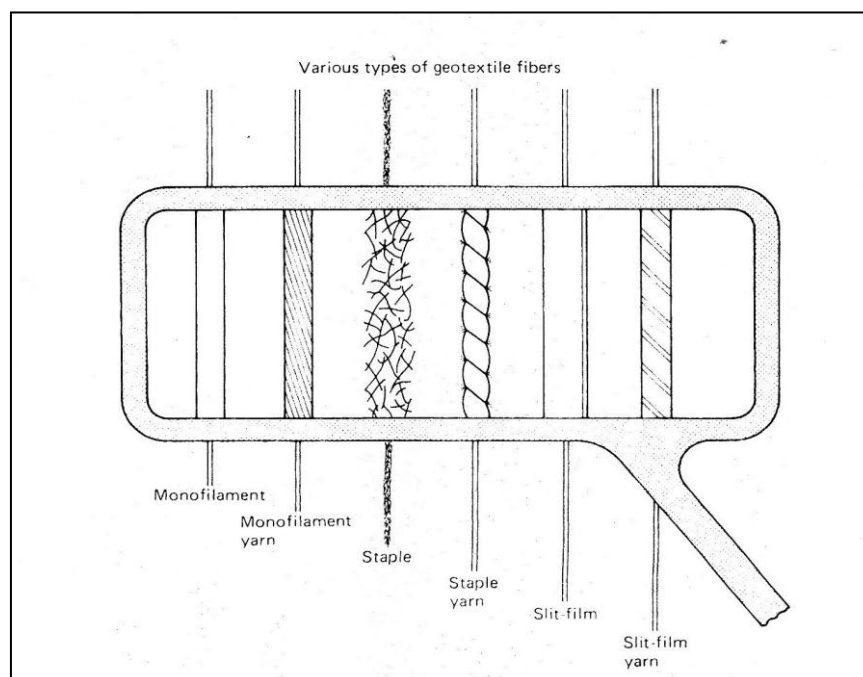


Figure 2.13: Types of fibers used in the manufacture of geotextiles (Koerner, 1986)

The vast majority of geotextiles are either woven or nonwoven due to their physical and mechanical properties which allow better performances in different applications. A number of typical woven and nonwoven geotextiles are in Figure 2.14. Woven geotextiles are manufactured from fiber, filaments, or yarns using traditional weaving methods and a variety of weave types. Nonwoven geotextiles are manufactured by placing and orienting the filaments or fibers onto a conveyor belt, which are subsequently bonded by needle punching or by melt bonding (Zornberg and Christopher, 2007). Figure 2.15 shows typical formation of woven and nonwoven geotextiles.

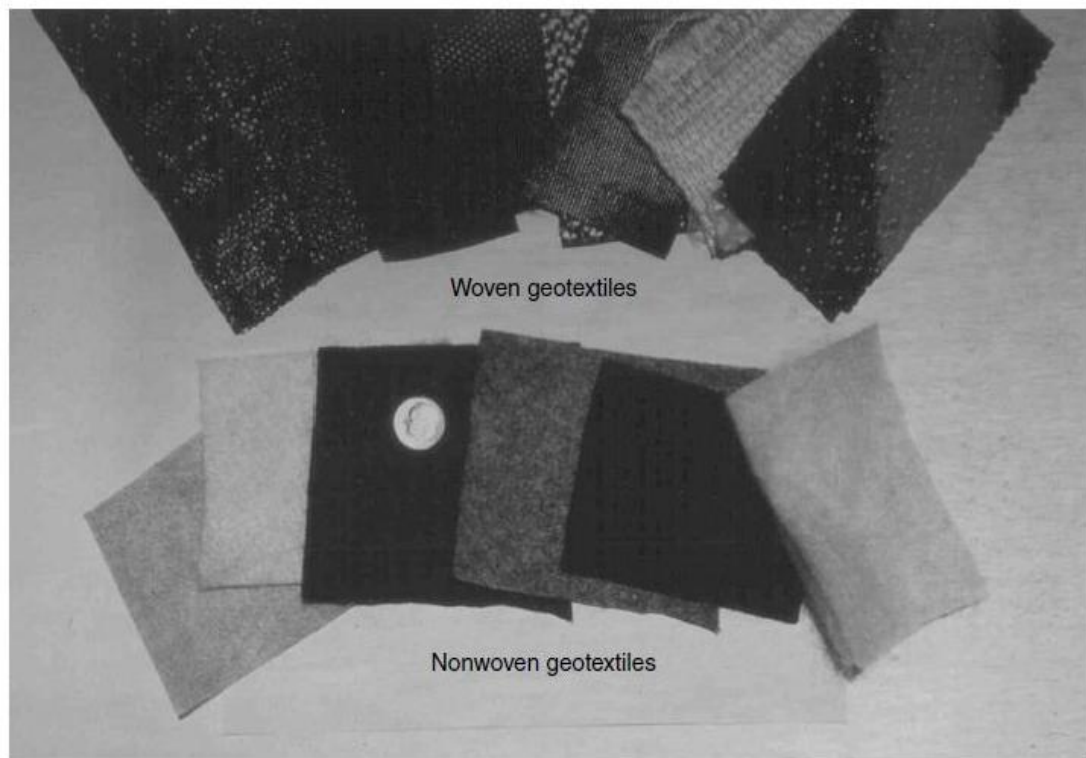


Figure 2.14: Typical woven and nonwoven geotextiles (Zornberg and Christopher, 2007)

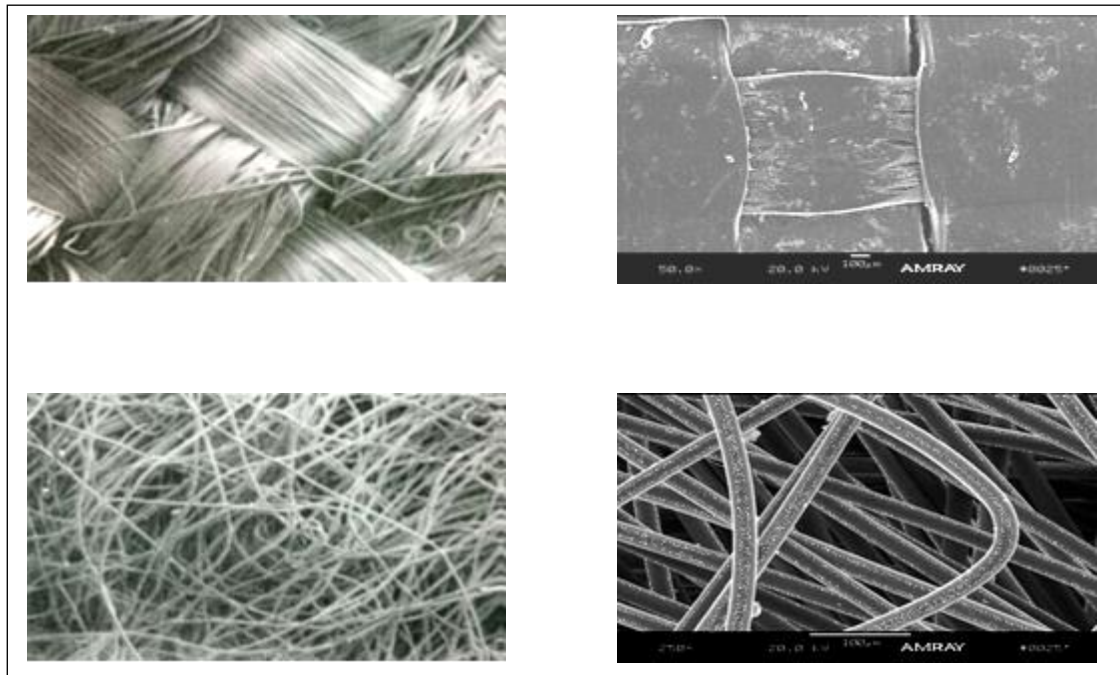


Figure 2.15: Microscopic view of woven (top two) and nonwoven (bottom two) geotextiles (Ingold and Miller, 1988)

2.6.2 Geogrids

According to ASTM (2003), Geogrid is a geosynthetic formed by a regular network of integrally connected elements with apertures greater than 6.35 mm to allow interlocking with surrounding soil, rock, earth, and other surrounding materials. Geogrids are primarily used for earth reinforcement and roadway stabilization. Nowadays, geogrids are extensively used in the construction of reinforced soil retaining walls. Figure 2.16 illustrates the interlocking mechanics of geogrid-soil composite.

Geogrids are mainly produced from polypropylene, polyethylene, polyester, or coated polyester. The use of polyester in manufacturing of geogrids is increasing because of its high strength and creep resistance (Table 2.3). The coated polyester geogrids are typically woven or knitted. These types of geogrids are generally known as flexible geogrids.

Coating is generally performed using PVC or acrylics to protect the filaments from construction damage and to maintain the grid structure. The polypropylene geogrids are either extruded or punched sheet drawn, and polyethylene geogrids are exclusively punched sheet drawn (Zornberg and Christopher, 2007). The extruded geogrids are usually called stiff geogrids which are divided into two categories; uniaxial and biaxial (Figure 2.17). Some of available geogrids are shown in Figure 2.18.

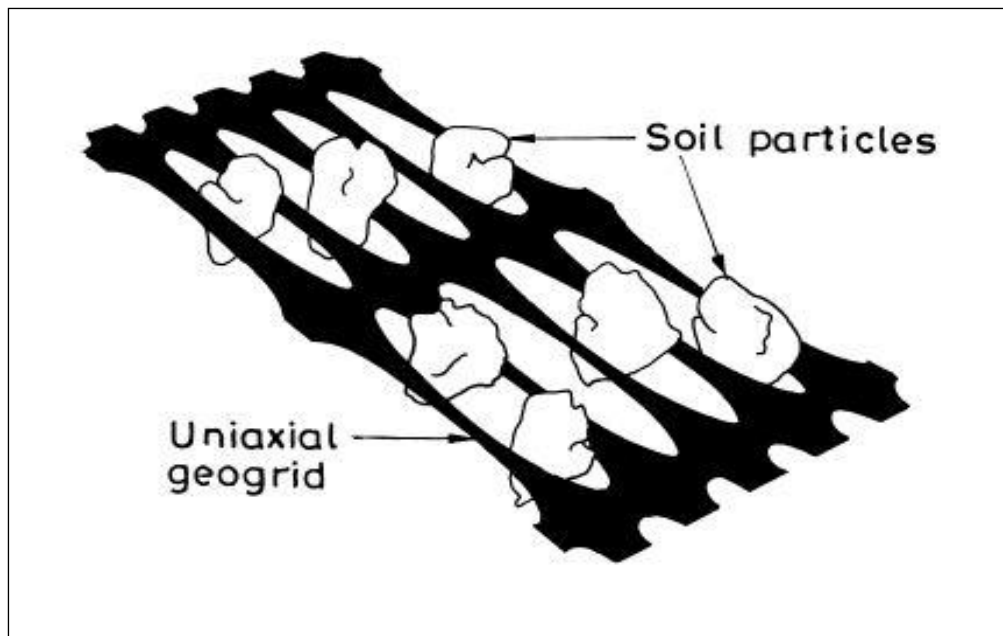


Figure 2.16: Interlocking behavior of geogrid reinforced soil (Shukla, 2002)

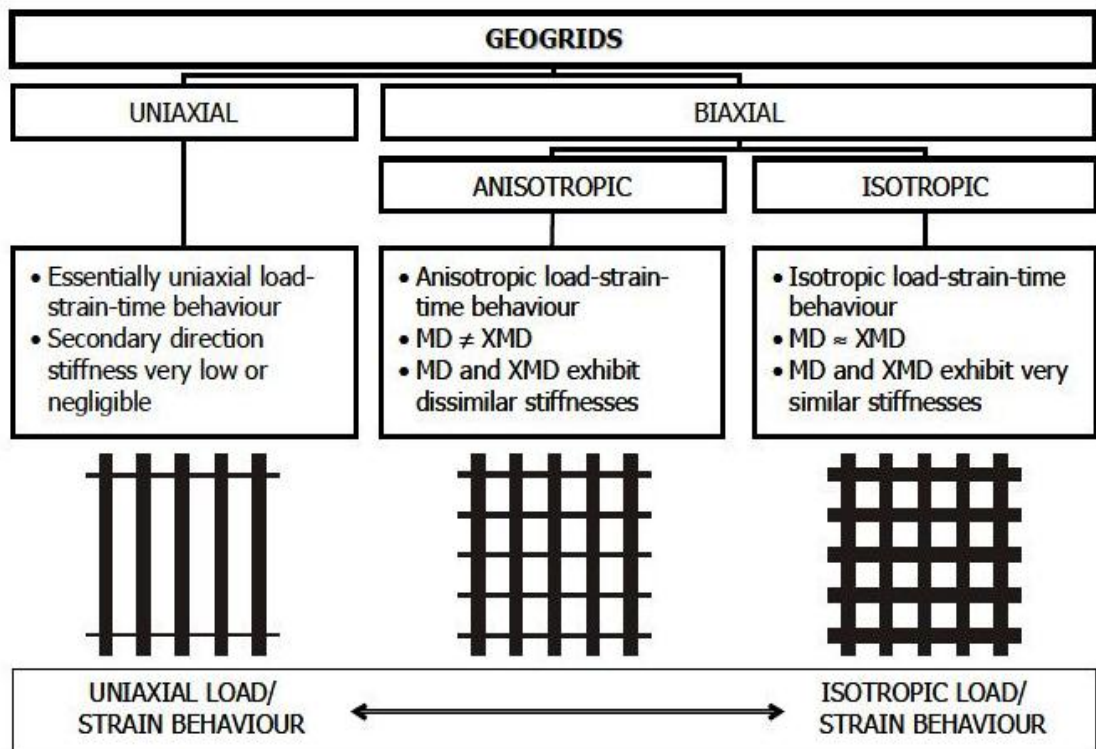


Figure 2.17: Various types of geogrids (McGown, 2009)

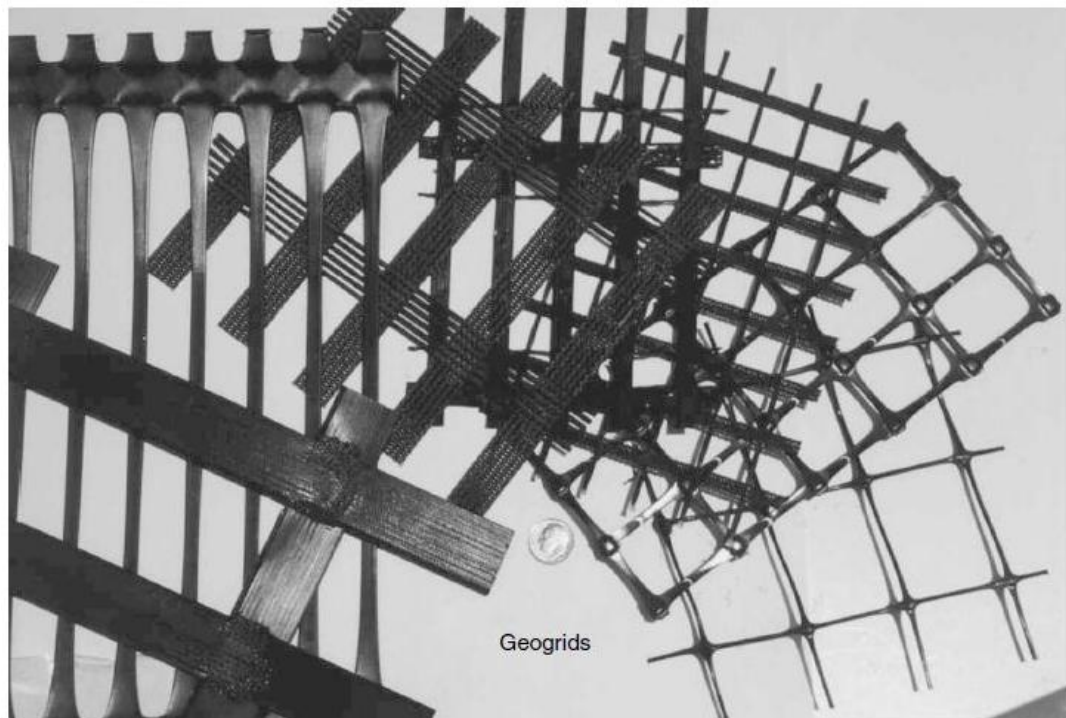


Figure 2.18: Typical geogrids (Zornberg and Christopher, 2007)

In geosynthetic reinforced segmental retaining wall systems, the following types of geosynthetics are widely used (Berg et al., 2009):

1. High Density Polyethylene (HDPE) geogrid. These are of uniaxial grids and available in different strengths.
2. PVC coated polyester (PET) geogrid. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. For longevity the PET is supplied as a high molecular weight fiber and is further characterized by a low carboxyl end group number.
3. High strength geotextiles made of polyester (PET) and polypropylene (PP) are used.

Figure 2.19 demonstrates typical strength behaviors of some geosynthetics used in reinforced soil structures. The geosynthetics (geogrids and geotextiles) used in this investigation have been discussed in details in Chapter 3.

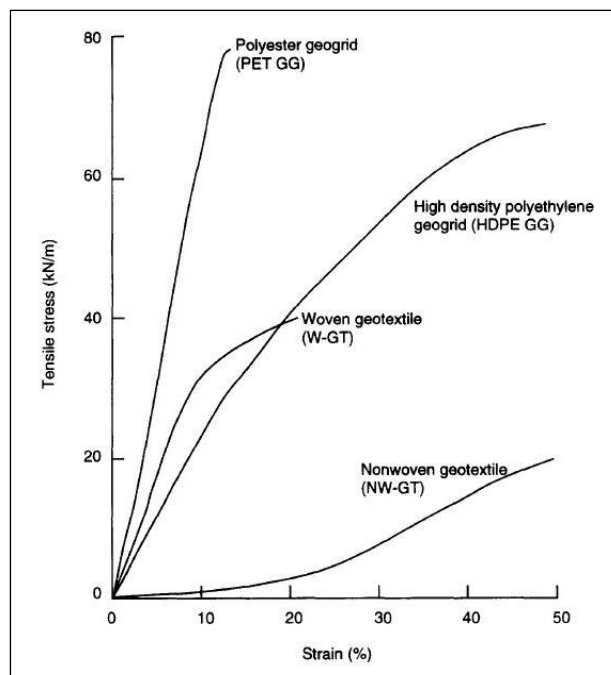


Figure 2.19: Typical tensile behaviors of some geosynthetics (Koerner and Soong, 2001)

2.7 Design methodology of GR-SRWs

For the analysis, design and construction of reinforced soil retaining walls, a number of guidelines have been developed, practiced, and modified; such as AASHTO (1996) Standard Specification for Highway Bridges, FHWA Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (Berg et al., 2009), NCMA Design Manual for Segmental Retaining Walls (Collin, 1997) and BS 8006 (1995) Code of Practice for Strengthen/Reinforced Soil and Other Fills. First three guidelines (AASHTO, FHWA and NCMA) are well established manuals used for the design of reinforced soil walls in North America (Collin, 2001). The third guidance NCMA is a most comprehensive design manual for segmental retaining walls which specially deals with GR-SRWs. Koerner and Soong (2001) reported that NCMA method is least conservative over FHWA method. An overview of design methodology is referred herein based on NCMA (Collin, 1997) guideline.

According to NCMA (Collin, 1997) design methodology, engineers have to pay attention on stability analyses related to four general modes of failure:

1. External stability
2. Internal stability
3. Local facing stability and
4. Global stability

2.7.1 External stability

External stability analyses examine the stability of the reinforced soil block (including the facing column) with respect to active earth forces generated by self-weight of the retained soils and distributed surcharge pressures beyond the reinforced zone.

The minimum length of geosynthetic reinforcement (L) is determined by checking base sliding, overturning, and bearing capacity failure modes (Figure 2.20). Collin (1997) recommends a minimum length of reinforcement is $0.6H$, where H is the height of wall.

2.7.2 Internal stability

Internal stability analyses study the performance of geosynthetic reinforcement used in reinforced soil zone and its effect on monolithic soil block. The minimum strength, number and spacing of the reinforcement layers are determined by examining tensile overstress, pullout, and internal sliding modes of failure (Figure. 2.21).

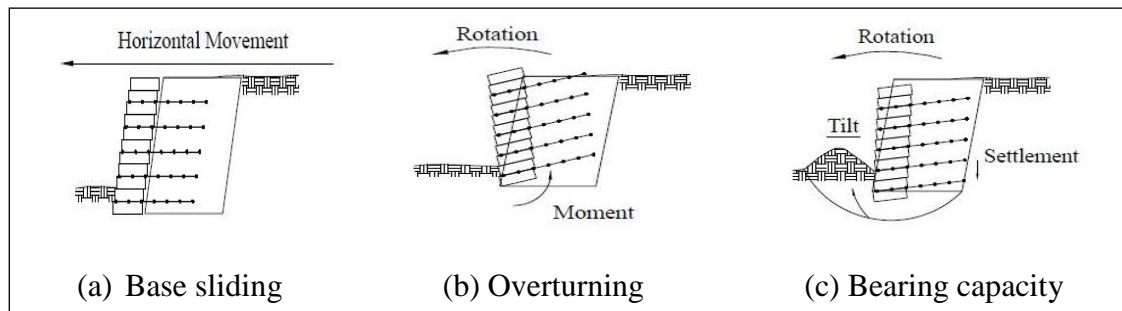


Figure 2.20: Main modes of failure for external stability (Collin, 1997; NCMA 2010)

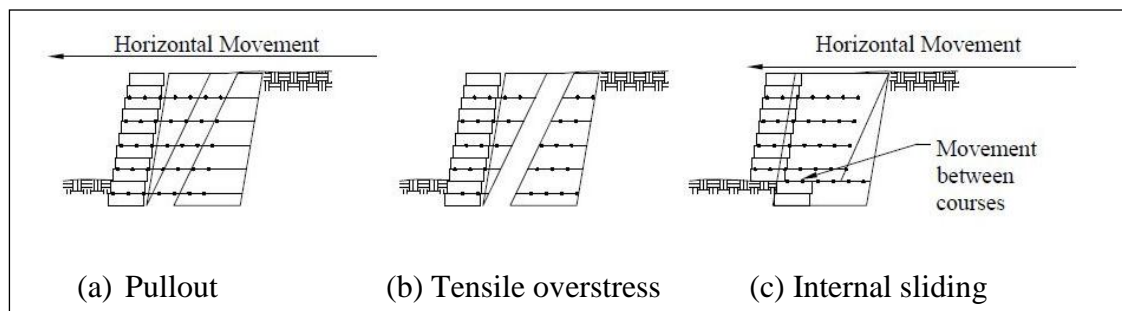


Figure 2.21: Main modes of failure for internal stability (Collin, 1997; NCMA, 2010)

2.7.3 Local facing stability

Local stability analyses deal with the column of facing units to ensure its intactness and limited deformation. The maximum vertical spacing of reinforcement is calculated by inspecting facing connection failure, bulging (shear) and maximum unreinforced height (Figure 2.22). Local stability is controlled by specific engineering performance properties of SRW units i.e. shear and connection strength.

The research study especially concentrates on interface shear capacity of a new block system. So a brief outline of bulging failure analysis is referred in this section.

2.7.3(a) Bulging

Bulging is the out of alignment of one or more layers of SRW units. It occurs when excessive earth pressure being applied at the back of facing column than shear resistance of the facing systems (Figure 2.23). Shear resistance of the blocks is influenced by the shear transferring device (Figure 2.8). Therefore, all units used in reinforced SRWs must possess sufficient interface shear capacity to counteract the horizontal earth pressure being applied between layers of geosynthetic reinforcement.

For bulging analysis, the dry-stacked column of SRW units are modeled as a continuous beam subjected with earth pressure (distributed load) and a simplified equivalent beam method is used to generate shear force along the wall. From the Figure 2.24, it is seen that the shear force applied to SRW units varies with location along the wall and the theoretical maximum shear forces occur at reinforcement elevations. The resistance to bulging is controlled by the magnitude of applied pressure, vertical spacing of reinforcement and interface shear capacity of modular blocks.

Interface shear capacity for a block system is evaluated by full scale laboratory test (performance test) (Figure 2.25). The Details of full scale laboratory study for the new block system is referred in Chapter 4.

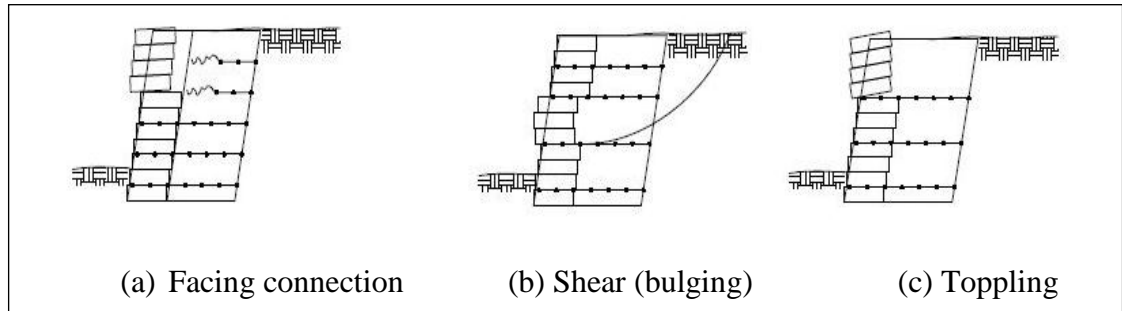


Figure 2.22: Main modes of failure for local facing stability (Collin 1997; NCMA, 2010)

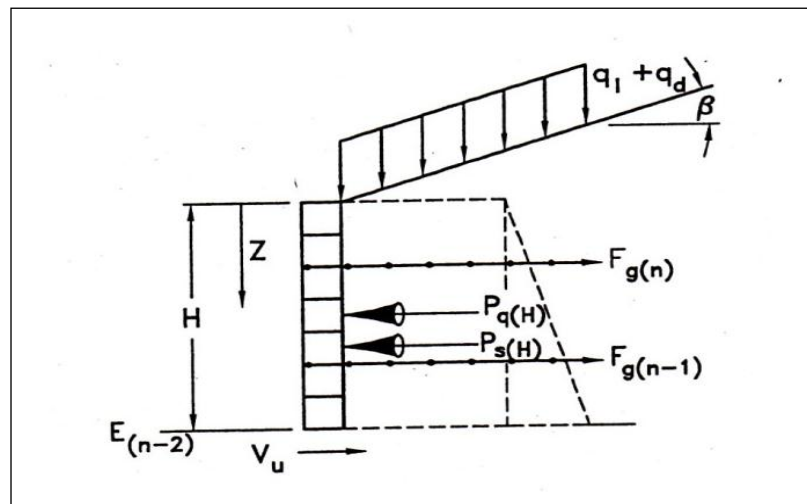


Figure 2.23: Shear force analysis for bulging (Collin, 1997)

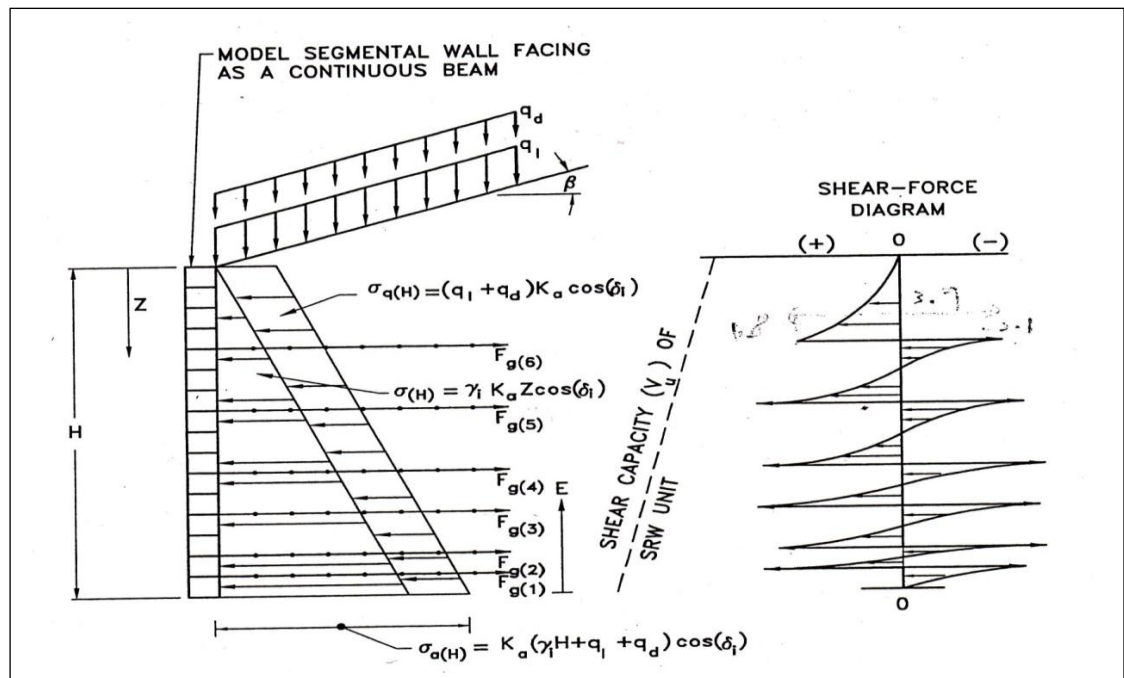


Figure 2.24: Typical shear force diagram and pressure distribution for GR-SRWs (Collin, 1997)

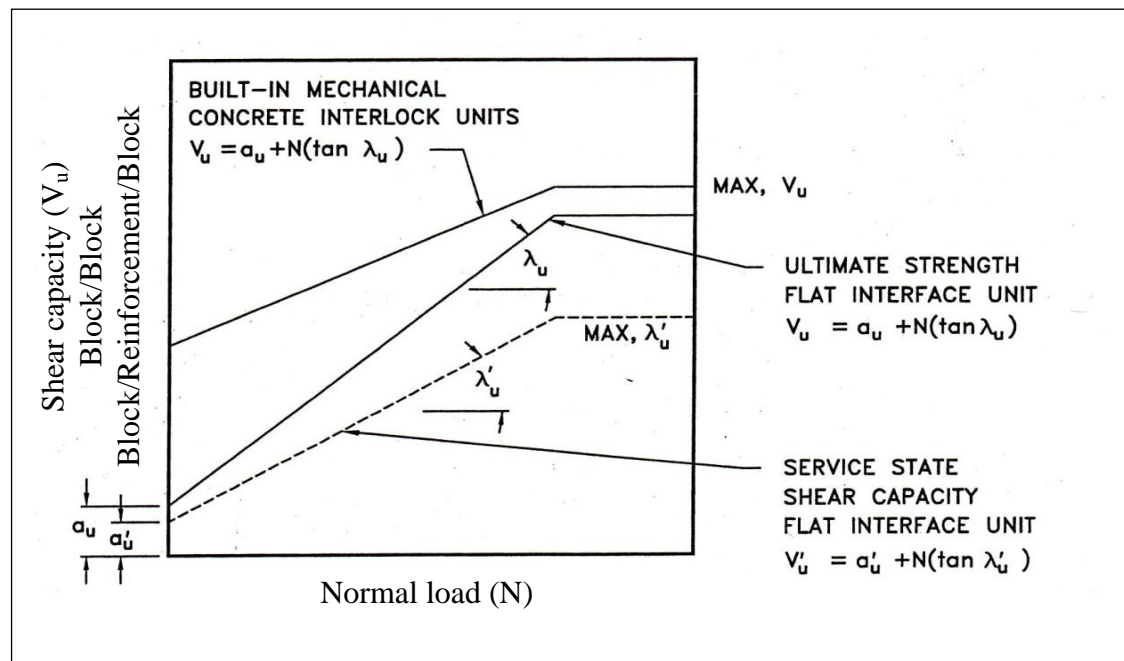


Figure 2.25: Typical shear capacity performance properties for SCUs (Collin, 1997)

2.7.4 Global stability

Global stability is the mass movement of the entire reinforced soil SRW structure including soil adjacent to the structure (Figure. 2.26). Generally, the reinforced soil SRW is assumed to act as coherent structure in the overall rotating mass. Over all stability is influenced by the surrounding structure and soil conditions. Details are found in NCMA (Collin, 1997).

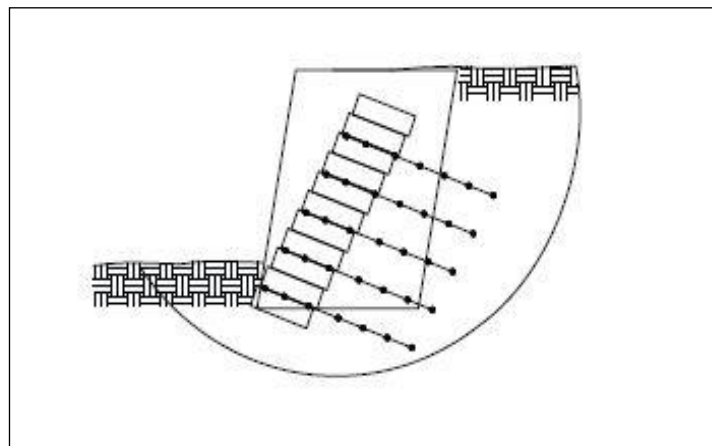


Figure 2.26: Global stability for GR-SRWs (Collin, 1997)

2.8 Previous related works

For full scale laboratory study of segmental concrete units, Bathurst and Simac (1993) originally developed a large scale apparatus in 1993 at the Royal Military College (RMC) of Canada. In the originally developed apparatus, a fixed vertical piston was used for applying surcharge load during testing. The authors also recommended a test procedure to compute the performance parameters of the connection tests. Later, the test method and apparatus were adapted by NCMA as a protocol for concrete block-geosynthetic facing connection testing.

Bathurst and Simac (1994) investigated interface shear strength for different types of concrete blocks with and without inclusion of geosynthetics. The interface shear tests were performed using a modified apparatus (fixed vertical piston with air bag) which was originally developed by Bathurst and Simac (1993). The concrete blocks used consisting of hollow core, shear key and tailing edge. Block to block interface peak shear capacity was determined for different combinations of concrete blocks. But the limited data set was unable to illustrate the efficient shear connection system. The authors also reported the inclusion of polyester geogrid reduced the interface shear capacity with respect to block to block shear.

Bathurst and Simac (1997) reported that shear key or connector increases the interface shear capacity. The authors also showed that the presence of a geosynthetic inclusion at the interface has a great influence on the interface shear capacity of the modular block system. It depends on the flexibility of geosynthetic reinforcements as well as block's interlocking system. The results concluded that relatively stiff geogrids (HDPE) decrease the interface shear capacity of a segmental unit system with a built in shear key. The authors also reported that the presence of flexible geogrids (geotextiles) also

increase the interface shear capacity of that block system. The increment of shear capacity resulted from the cushion effects of the flexible geosynthetic situated at the block's interface. Huang et al. (2007) also reported the effect of interface shear stiffness on the performance of reinforced soil retaining walls.

Natural coarse aggregates are expansively used in the different fields of civil engineering constructions. In the recent times, the use of fresh aggregates as filling materials for segmental retaining walls has increased extensively. Bathurst and Simac (1993, 1994 and 1997) used crushed stone (fresh aggregates) as infill for hollow block systems to provide positive interlocking between the courses (Bourdeau, 2001). The use of natural aggregates is unsustainable (extinction of natural resources) and expensive. Touahamia et al. (2002) investigated the frictional performance of recycled materials with and without reinforcement. The authors recommended that recycled aggregates (crushed concrete) could be used as an alternative of natural aggregates for filling purposes where the strength requirement is not an issue of concern.

Bathurst et al. (2008) evaluated the effect of normal loading arrangement of interface shear testing. Three different types of normal loading arrangement were investigated in the study: (1) fixed vertical piston, (2) adjustable vertical piston, and (3) flexible airbag. Four different types of dry cast block (hollow and solid) system were selected and used for this investigation because of their different shear transferring devices such shear key, tailing lips and shear pins. The remaining one was purely frictional solid block system without any shear transferring device. The results of investigation reported that vertical loading arrangement greatly influences the interface shear capacity of the block systems which show dilatant behavior (block with shear or tailing lips).

A fixed vertical piston increased the normal load during interface shear testing because of its out of vertical movement. Among three types of vertical loading arrangements, flexible air bag arrangement kept the normal loading constant during interface shear testing for all types of block systems. Flexible air bag arrangement is complex and time consuming than other proposed normal loading arrangements.

Astarci (2008) reported frictional behavior of connection tests between hollow facing blocks and geosynthetics with different combination of infill materials. Two types of geogrids were used that manufactured from polyester and polypropylene. Woven geotextiles were also used which made of polypropylene. Sand and gravel were used as infill. Connection tests were performed under three different normal loads and tensile loads were applied by dead weights acting on hanger arrangement. So loading rate was not controlled as per NCMA design method. Tensile stress vs. normal stress curve was outlined to find out angle of friction between blocks and geotextiles. From the investigation it was found that gravel-geotextile combination showed higher angle of friction over sand-geotextile combination. The same frictional behavior also found by Selek (2002). Astarci (2008) also reported the angle of friction of gravel-geogrid combination was higher than all other combinations. The angle of friction of gravel-geogrid combination was around 64 to 67 degree. The biaxial polyester geogrid gave more internal friction over uniaxial extruded geogrid composed of polypropylene. From this study it can be concluded that gravel-geogrid combination increases the connection strength of reinforced segmental walls because of the interlocking mechanism.

2.9 Summary of key points

Based on a comprehensive review over the past works, the major points/observations can be summarized as follows:

1. Performance parameters (shear and connection) of modular concrete units can only be obtained by full scale laboratory or field tests.
2. Performance tests can be done using a specially designed apparatus which is capable of applying horizontal (pull/push) and vertical (normal) load simultaneously.
3. Vertical load (surcharge) is applied by a hydraulic piston/actuator and its loading arrangement greatly affects the interface shear testing of segmental retaining wall units.
4. The strength properties (shear and connection) are influenced by the geometry and type of shear transferring device such as continuous keys, lips, dowels or pins.
5. The research works had mainly been focused on the effect of shear key on the frictional performance of segmental concrete units although mechanical shear connector has great influence on interface shear strength. The understanding about the effect rigidity of shear pins on the interface shear capacity is not clear yet.
6. Hollow segmental units provide better interlocking among the course while the cavities are filled with granular materials. As granular in-fills crushed stone aggregates were used without considering the use of recycled aggregates.
7. Geosynthetic inclusion at the block interface significantly influences the interface shear capacity and it depends on the structure, thickness and polymer type.

CHAPTER 3 MATERIALS

3.1 General

This chapter presents the properties of the materials used in the full scale laboratory study of the facing units. The materials included in this chapter are segmental concrete unit, granular infill, shear connector, and geosynthetic reinforcement.

3.2 Segmental concrete unit

A newly designed segmental unit system is used in this research. The innovated concrete unit is named as I-Block due to its geometrical shape (Figure. 3.1). The I-Blocks are wet cast masonry units made from 30N/mm^2 concrete, which consist of one center web and a tail/rear flange that is extended beyond the web. The rear flange is tapered to allow the blocks to form curve walls. I-Blocks are flat interface modular concrete blocks, which can be stacked with and without shear connectors. The maximum tapered angle of the I-Block is approximately 11.3° . I-Blocks are double open-ended units and provide a larger hexagonal hollow space in conjunction with two units, and the equivalent hole dimensions are about 450 mm in length, 257 mm in average width and 300 mm in height. Thus, I-Block promotes the increment of wall face area and also minimizes the use of concrete volume. The infilled weight of the block varies approximately 88 to 95 kg according to the unit weights of the granular in-fills used in this investigation.

I-Block offers virtually any type of wall face patterns desired and provides a more efficient use of construction material regardless the technical aspect of a sound engineering retaining wall system. I-Blocks could also be stacked and reinforced to form an aesthetically pleasant looking reinforced concrete wall (Figure 3.2).

I-Blocks in this case are served as a hollow block formwork system for reinforced concrete wall casting. This system will thus eliminate the need of using wall face steel bars, which is usually placed to control the cracks at the wall face in the conventional reinforced concrete wall. Hence, I-Block is, in fact, a two-in-one block system, which offers an aesthetically pleasant looking and cost-effective wall system. The blocks are supplied and produced by Soil & Slope Sdn. Bhd. in Malaysia. ASTM protocol is implemented followed to find out properties of the blocks. Table 3.1 summaries the physical and mechanical properties of the blocks.

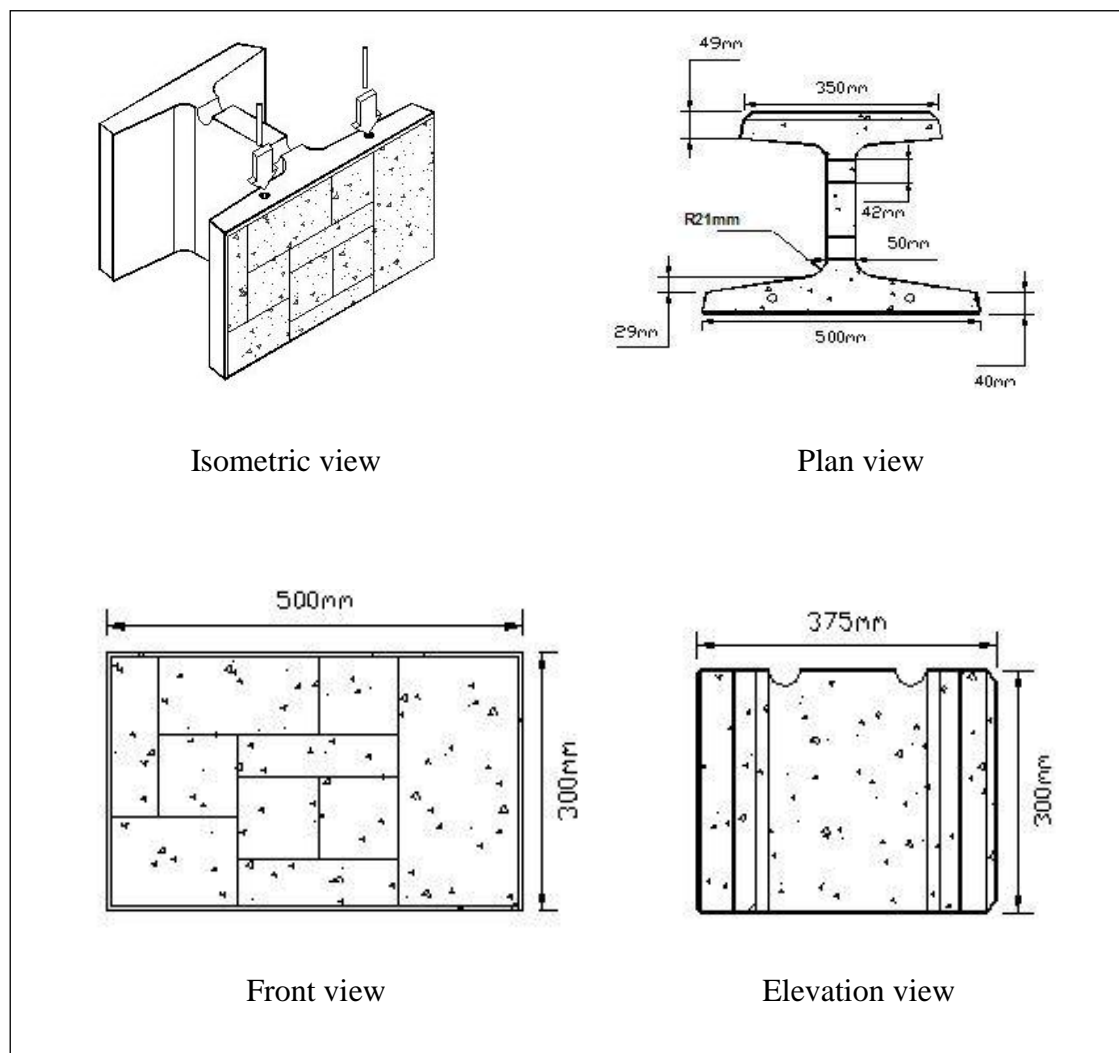


Figure 3.1: Details of innovated I-Block (courtesy of Soil & Slope Sdn. Bhd.)

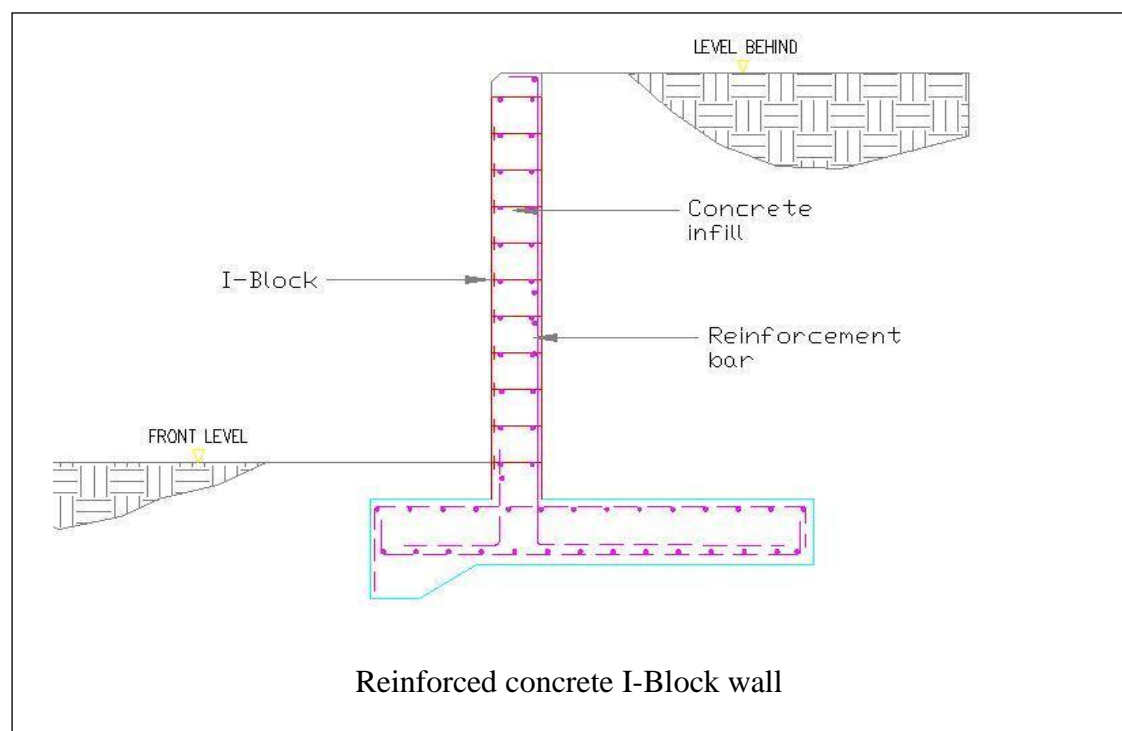
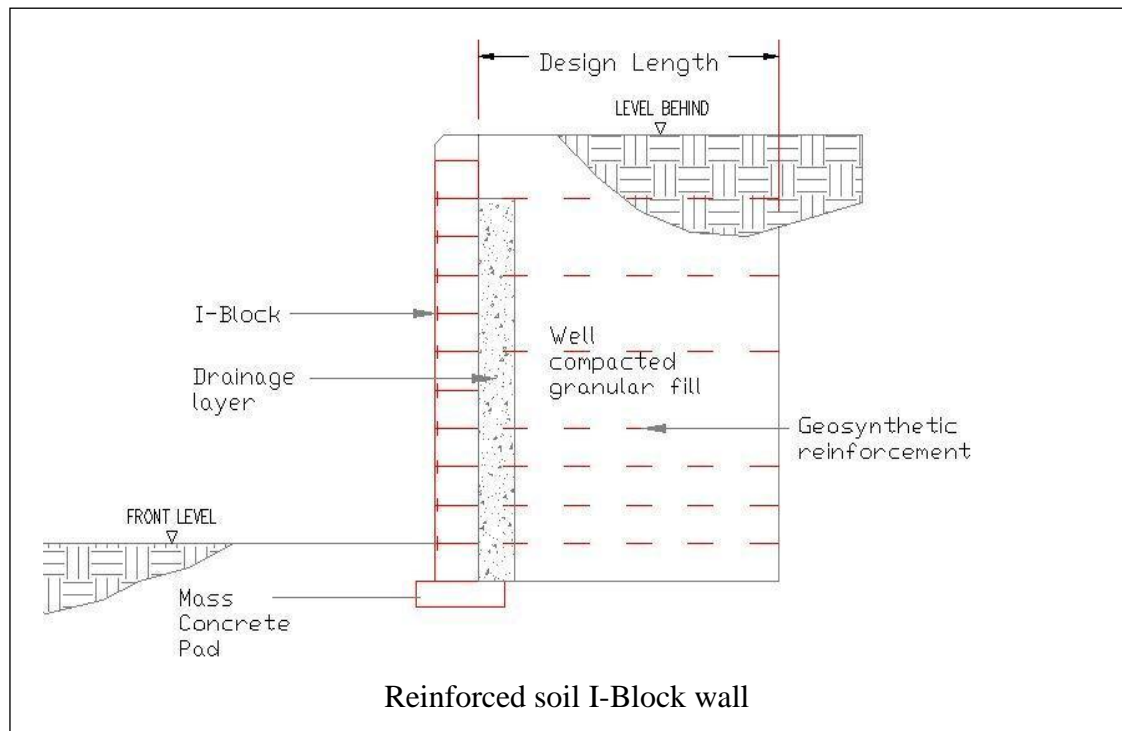


Figure 3.2: Different applications of I-Blocks showing details drawing of installation (courtesy of Soil & Slope Sdn. Bhd.)

Table 3.1: Physical and mechanical properties of I-Block

Property	Value
Dimensions ($W_u \times H_u \times L_u$) ^a in mm	375x300x500
Weight (kg)	41-42
Oven dry density (kg/m^3)	2166
Water absorption capacity (%)	7.1
Moisture content (%)	3.7
Net compressive strength (MPa)	8.0

^a W_u = Width (Toe to heel), H_u = Height, L_u = Length (Parallel to the wall face)

3.3 Granular infill

Three (3) different types of coarse aggregates are used in this current series of tests as granular in-fills. The hollow cores between the blocks are infilled with natural coarse aggregate (NCA) and two (2) different types of recycled concrete aggregates (RCAs). Recycled aggregates are produced from 30 grade normal concrete (RCA 1) and 60 grade palm oil fuel ash (POFA) concrete (RCA 2). The broken and tested “I” blocks are used as source of recycled aggregate (RCA 1). On the other hand, tested and spared POFA concrete cylinders are utilized as raw material of RCA 2. Natural (fresh) aggregates are 100% crushed limestone aggregates, which is collected from an aggregate supplier. Recycled aggregates are produced in concrete lab, manually using hammer. The maximum and nominal maximum sizes of the aggregates are 25 and 19 mm respectively. The particle size distribution of the granular in-fills meets the required ASTM standard size #57 gradations (ASTM D448-03a, 2003). Figure 3.3 shows the gradation curve. The physical properties of the in-fillers are given in Table 3.2. Figure 3.4 views the photographs of aggregates.

Table 3.2: Physical properties of granular in-fills

Property	NCA	RCA 1	RCA 2
Bulk density ^a (Kg/m ³)	1527	1336	1410
Specific gravity ^a	2.63	2.42	2.48
Water absorption (%)	0.48	5.51	3.70
Void content ^b (%)	42	45	43
Alkalinity (pH)	9.30	8.76	11.42
Uniformity coefficient, C _u	1.69	2.22	1.83
Coefficient of curvature, C _c	1.00	1.32	1.10
Fineness Modulus (FM)	7.16	6.82	7.47

^aSaturated surface dry; ^bOven dry

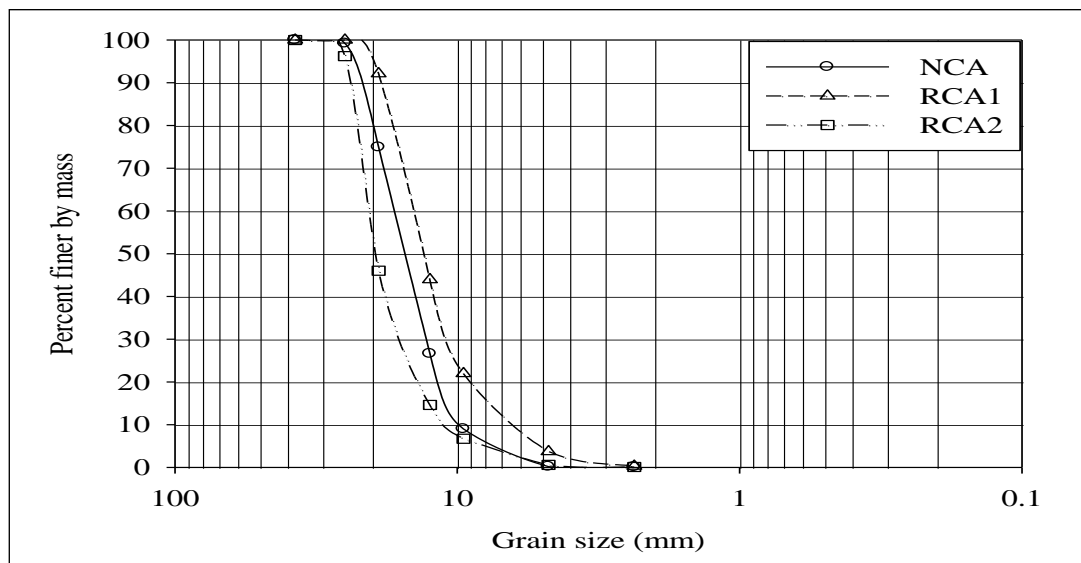


Figure 3.3: Grain size distribution curve for in-fillers

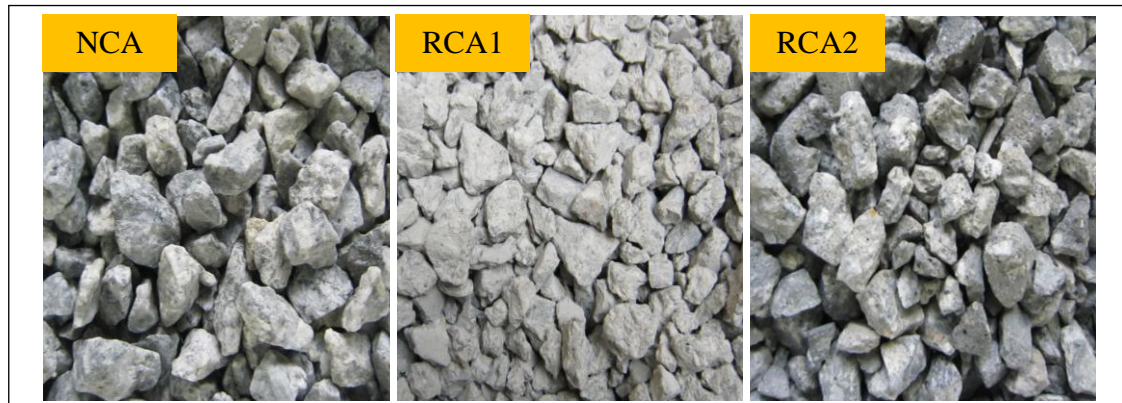


Figure 3.4: Photographs of granular in-fillers

3.4 Shear connector

Two (2) different types of shear connectors are used in this research. Steel and plastic shear pins are chosen because of their rigidity. Galvanized mild steel round bars are selected in the study as rigid mechanical connectors. According to the pin hole dimensions of the segmental concrete units, 12 mm diameter bars are selected, and the bars are cut into 125 mm in length. The physical and mechanical properties of the used round steel bars are illustrated in Table 3.3.

Ultrahigh molecular weight polyethylene (UHMWPE) plastic bars are used in this investigation as flexible connectors because of its toughness and flexibility. UHMWPE has also highest impact strength. The mechanical properties of UHMWPE were tested by Kromm (2003). White color UHMWPE round bars of 13 mm diameter are used, which is available in the market and the parent bars of 1 m in length are cut into 100 mm in length. The properties of the plastic bars are given in Table 3.4. Figure 3.5 shows the photographs of the shear pins.

Table 3.3: Physical and mechanical properties of steel bar

Property	Value
Yield strength (MPa)	347
Modulus of elasticity (MPa)	200,000
Elongation (%)	34
Density (kg/m ³)	7,850
Cross section area (mm ²)	113.10

(courtesy of AM Steel Mills Sdn. Bhd.)

Table 3.4: Properties of plastic bar

Property	Value
Yield strength at 23°C (MPa)	22
Modulus of elasticity (MPa)	750
Elongation at break (%)	>300
Charpy impact strength, (kJ/m ²)	No break
Melting point (°C)	135
Density (kg/m ³)	940
Cross section area (mm ²)	127.66

(courtesy of KHQ Industrials Supplies)



Figure 3.5: Photographs of Plastic (white color) and Steel (silver color) shear pins

3.5 Geosynthetic reinforcement

In this investigation, three (3) types of geosynthetic reinforcements are chosen: a knitted polyester (PET) geogrid (flexible), a high density polyethylene (HDPE) geogrid (stiff), and a non-woven polyester geotextile (flexible) those which are commonly used in Malaysia for GR-SRW constructions. The reinforcements are selected because of their high strength and low creep. Details of the reinforcements are referred as below:

3.5.1 Geogrid

Geogrid 1 is a knitted uniaxial geogrid prepared from high tenacity polyester yarns, and covered with a black polymeric coating. The major characteristics are good connection capacity with modular blocks and excellent interface friction behavior, and high tensile strength at low creep. It is widely used in the field of reinforced earth structures, bridge abutments, pile embankment, subgrade stabilization, railways, and slope reinforcement. A summary of the properties of Geogrid 1 provided by manufacturer is contained in Table 3.5. The dimensions and photograph of Geogrid 1 are indicated in Figure 3.6.

Geogrid 2 is an extruded uniaxial geogrid with elongated apertures and made from high density polyethylene (HDPE). The primary characteristics are good creep performance with low strain and high tensile strength under constant load, and it also provides good gripping capacity with the shear connectors of the modular block units. Geogrid 2 is mainly used for reinforcement of modular block walls, earth walls, slopes and bridge abutments.

Table 3.6 outlines the general properties of Geogrid 2 reported by manufacturer. Figure 3.7 demonstrates the typical dimensions and photograph of Geogrid 2.

Table 3.5: Basic properties of Geogrid 1

Property		Unit	Value
Short term tensile strength (T_c)	MD	kN/m	80.0
Short term tensile strength (T_c)	CD	kN/m	30.0
MD Tensile strength	2% strain	kN/m	16.0
MD Tensile strength	5% strain	kN/m	34.0
Strain at MD tensile strength		%	11.0
Creep limited strength at 120 years		kN/m	55.2
Weight		kg/m ²	0.32
Surcharge height limitation		m	8.7
Aperture size	MD	mm	23
Aperture size	CD	mm	21
Strand width	MD	mm	4.0
Strand width	CD	mm	3.0
Thickness		mm	1.40

Note: MD = machine direction; CD = Cross-machine direction. Unless noted otherwise, data are from manufacturer's literature (courtesy of TenCate Geosynthetics Asia Sdn. Bhd.)

Table 3.6: General properties of Geogrid 2

Property		Unit	Value
Short term tensile strength (T_c)	MD	kN/m	90.0
Short term tensile strength (T_c)	CD	kN/m	-
MD Tensile strength	2% strain	kN/m	23.7
MD Tensile strength	5% strain	kN/m	45.2
Strain at MD tensile strength		%	11.5
Creep limited strength at 120 years		kN/m	36.1
Weight		kg/m ²	0.55
Surcharge height limitation		m	6.9
Nominal distance between two bonds(P_{nom})		mm	258
Distance between two ribs (A_{td})		mm	16
Bond thickness (T_b)		mm	4.1
Rib thickness (T_r)		mm	1.1
Bond width (B_w)		mm	18
Strand width (S_w)		mm	6

Note: Unless noted otherwise, data are from manufacturer's literature (courtesy of Qingdao Etsong Geogrids Co., Ltd.)

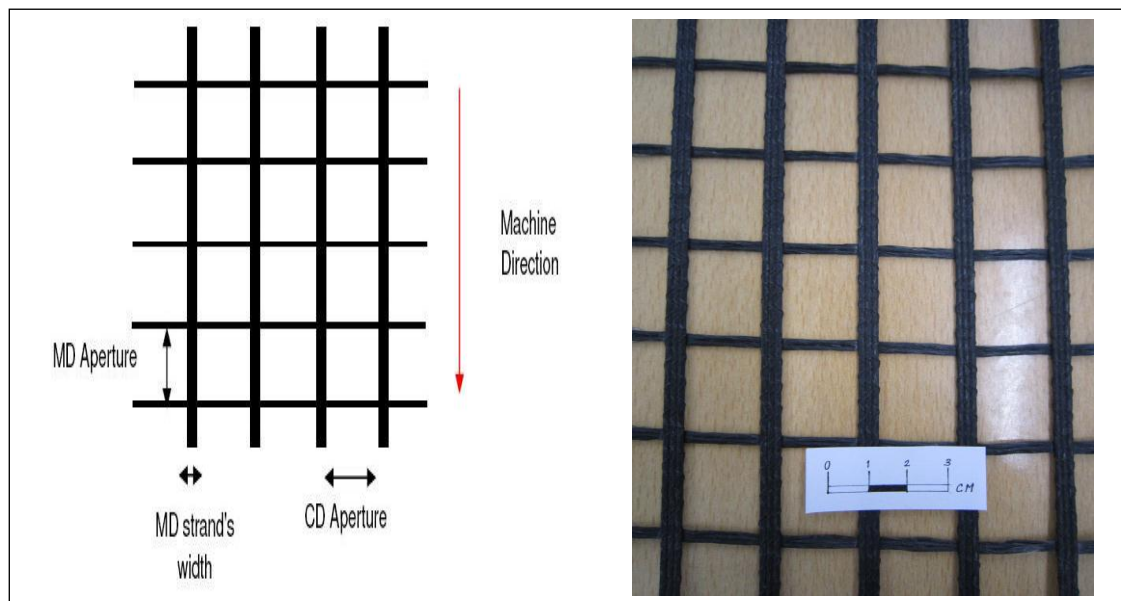


Figure 3.6: Typical dimensions and photograph of Geogrid 1 (courtesy of TenCate Geosynthetics Asia Sdn. Bhd.)

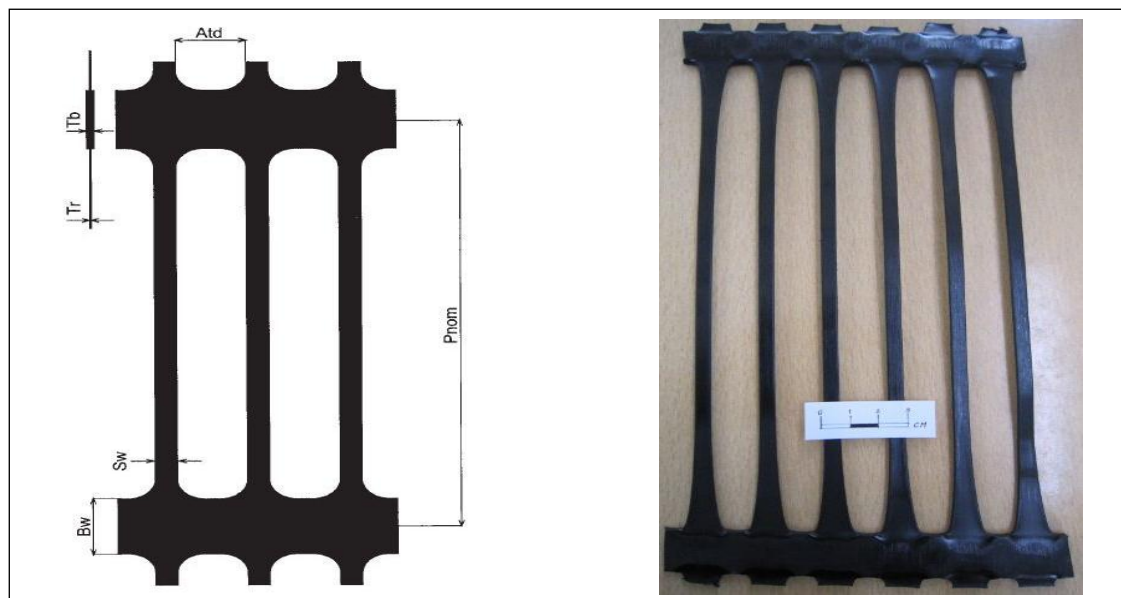


Figure 3.7: Typical dimensions and photograph of Geogrid 2 (courtesy of Qingdao Etsong Geogrids Co., Ltd.)

3.5.2 Geotextile

A non-woven needle punched uniaxial composite geotextile was used, which consisting of combination between high tenacity polyester yarns stitched to polypropylene continuous filaments. It is characterized by high tensile strength at low elongation and by high water flow capacity in its plane. Typical application areas of this geotextile are retaining wall, reinforced steep slopes, parking area stabilization, and foundation cushioning. The physical and mechanical properties of the geotextile are presented in Table 3.7. A photograph of the geotextile is shown in Figure 3.8.

Table 3.7: Physical and mechanical properties of Geotextile

Property		Unit	Value
Short term tensile strength (T_c)	MD	kN/m	75.0
Short term tensile strength (T_c)	CD	kN/m	14.0
MD Tensile strength	2% strain	kN/m	12.5
MD Tensile strength	5% strain	kN/m	33.8
Strain at MD tensile strength		%	10.0
Creep limited strength at 120 years		kN/m	45.0
Water flow rate normal to the plane		mm/s	65
Weight		kg/m ²	0.34
Surcharge height limitation		m	8.7
Thickness		mm	2.2

Note: Unless noted otherwise, data are from manufacturer's literature (courtesy of Polyfelt Asia Sdn. Bhd.)

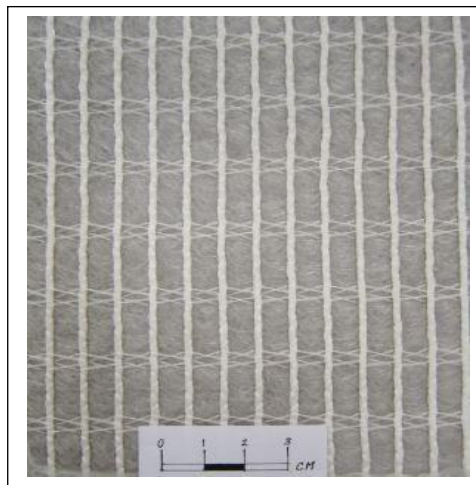


Figure 3.8: Photograph of Geotextile

CHAPTER 4 APPARATUS, INSTRUMENTATION AND TEST PROGRAM

4.1 General

This chapter describes the apparatus developed at University of Malaya to perform full scale laboratory study of segmented retaining wall units with a view to providing a brief overview on the instrumentation and data acquisition system. In addition, this chapter also includes a generic test procedure for different test groups.

4.2 Design and development of apparatus

4.2.1 Background

Segmental block systems are used in different fields of civil engineering, especially in various areas of geotechnical engineering. Segmental concrete blocks are discrete in nature and its stability (facing) is an important issue in the current design guidelines of segmental retaining walls and may have effect on internal stability of SRW systems (Bathurst and Simac, 1997). Facing stability is mainly controlled by performance parameters (shear and connection strength). These parameters are evaluated only by full scale laboratory or field tests of blocks system used in segmental retaining walls. Today, a variety of blocks are available and used with different types of connection systems. Details have been described in Section 2.4 of Chapter 2. To find out the performance parameters according to ASTM and NCMA protocols, it is need to design and develop a suitable set facility which is well-suited for all types of block systems, and closes enough to simulate actual field condition.

A typical test apparatus for full scale laboratory study of SRW units was designed and developed by Bathurst and Simac (1993) and later on adopted in ASTM and NCMA standard guidelines.

Since, the apparatus is not a standard one, so there is always a choice to modify and redesign it according to the user's block systems and available technologies (Thiele, 2005; Guler and Astarci, 2009). But the performance tests need to be done according to the standard guidelines' requirements.

After reviewing the NCMA SRWU-1 (1997), NCMA SRWU-2 (1997), ASTM D 6916 (2006) and ASTM D 6638 (2001) test protocols, it was found that protocols recommend a fixed vertical actuator with roller or airbag arrangement. Bathurst et al. (2008) reported that normal loading arrangement greatly influences the performance parameters of different block systems. From the investigation, it was concluded that fixed vertical actuator with flexible airbag arrangement provides better loading arrangement that keeps the normal load constant over the period of shear testing. But the use of flexible airbag is more cumbersome and time-consuming test arrangement. By considering the vertical loading arrangement and the block system used in this study, the test apparatus was modified and redesigned in terms of vertical and horizontal loading assembly, capacity and clamping systems. The details about test facility are given in the following section.

4.2.2 Description of the modified apparatus

The apparatus was designed and developed at University of Malaya (UM) to satisfy the ASTM and NCMA criteria for full-scale laboratory testing of segmental concrete units. It is a modified large-scale direct shear box apparatus with connection testing facility for modular block units.

The apparatus used for this study consists of two major parts: loading structure which applies load on testing sample as well as provides support to testing setup and electric pump system acts as a load source and provides vertical and horizontal load on test sample simultaneously. Figure 4.1 illustrates the photograph of modified apparatus at initial stage. Further modifications were added later on to carry out the tests properly.

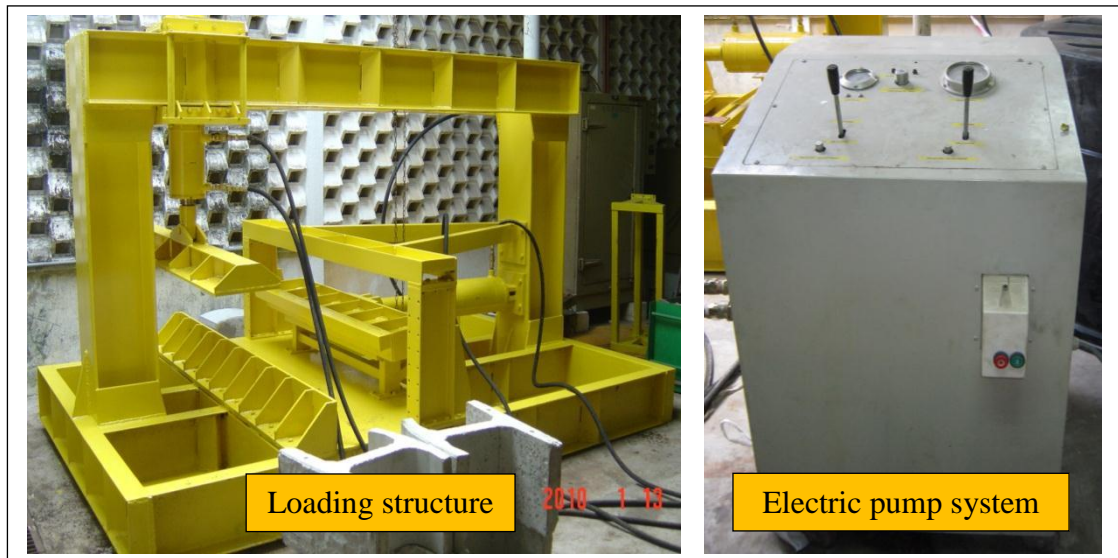


Figure 4.1: Photograph of test apparatus

4.2.2.1 Loading structure

Loading structure is the basic part of testing device as shown in Figure 4.2. The key components of the loading structure are as follows:

- Loading frame
- Restraining plate
- Vertical piston/actuator
- Vertical loading platen
- Horizontal piston/actuator
- Shear loading plate
- Geosynthetic clamping assembly for interface shear tests
- Tensile loading clamp and assembly

4.2.2.1(a) Loading frame

Loading frame is the skeleton (frame structure) of the apparatus that provides a platform for testing setup and support the other assemblies such as actuators, platens, clamping device and guide frame etc. The width of the platform is about 2000 mm that support a long base course of segmental units. The frame is capable to withstand high reaction forces developed by the vertical and horizontal actuators/pistons. The frame capacity is approximately 598 kN (60 ton) of normal load (surcharge) and 598 kN (60 ton) of horizontal load (shear or pullout). The heavy weight loading frame is leveled and anchored with rigid concrete floor to make it free from any inclination and vibration problems which may hamper the tests.

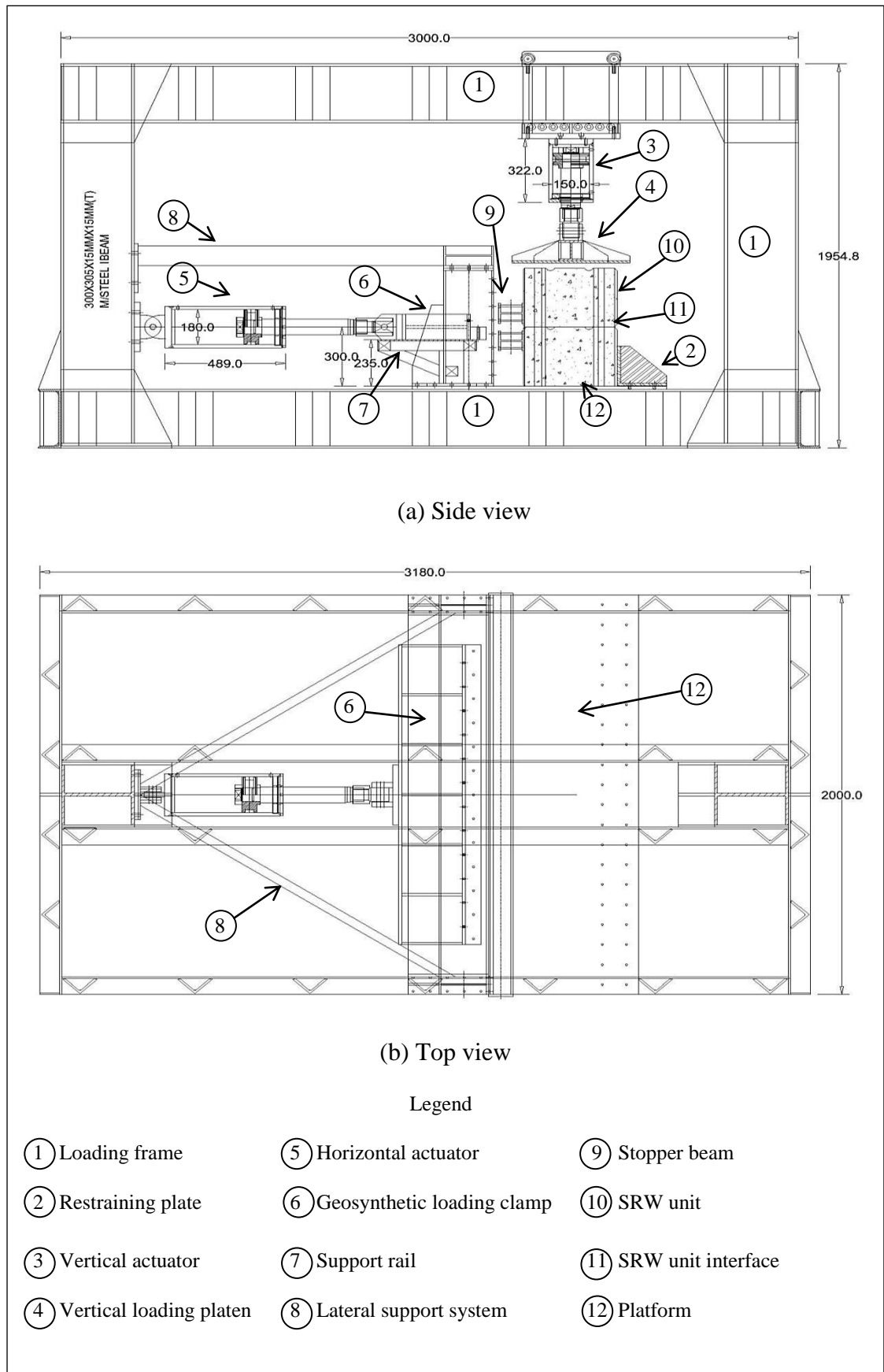


Figure 4.2: Schematic of test apparatus showing connection testing arrangement

4.2.2.1(b) Restraining plate

A rigid restraining plate of 200 mm in height is screwed to the platform of the apparatus to prevent horizontal movement of the base layer of concrete units during shear testing. In the current apparatus, height of the restraining plate is chosen according to the used I-Block size, which is $2/3^{\text{rd}}$ of block's height (300 mm). The position of the restraining plate can easily be adjusted using bolt embedment to accommodate different size of concrete blocks to be tested.

4.2.2.1(c) Vertical actuator

A double-acting hydraulic cylinder is used as a vertical actuator that applies normal or surcharge pressure through a loading platen over the stacked blocks. Hydraulic cylinder is mechanical actuator that converts fluid energy into indirection force through linear movement of piston. Double-acting hydraulic cylinders provide both pull and push loads, and also better for fast retraction. A vertical actuator of 379 kN (38 ton) push (advance) capacity is mounted with the loading frame using steel rollers to allow movement of topmost block layer during shear testing (Figure 4.2). Pull (retract) capacity of the cylinder is 269 kN (27 ton) at 21 MPa working pressure. Cylinder bore diameter of the actuator is 150 mm and it is capable of applying 129 mm stroke. Clevis joint is used to connect the plunger of 80 mm in diameter with loading platen.

4.2.2.1(d) Vertical loading platen

A rigid rectangular steel plate is used as loading platen for distributing normal pressure uniformly to the top of concrete blocks through stiff rubber mat. Initially, a loading platen of 1500 mm in length and 300 in width was jointed with the vertical ram.

It was then changed to 480 mm wide platen of equal length (1500 mm) because of I-Block's width (370 mm).

The wider platen can easily distribute surcharge load over the top of segmental units without using of any bearing plate in between platen and top surface of block. This wider platen can also be used to test wall more than 1 m (1000 mm) in length. Loading platen is pinned with clevis eye of the plunger. The clevis joint allows flexibility and suppleness to the platen for sitting over the top concrete blocks freely. Loading platen also may be changed according to the user's choice for testing different types of modular blocks.

4.2.2.1(e) Geosynthetic gripping clamp

A steel gripping clamp is designed to grip geosynthetics used at the interface for interface shear testing of segmental units. Figure 4.3 shows the details about the gripping clamp. The clamp is installed to the platform of loading frame using screws, which holds the geosynthetics from the back of blocks. Geosynthetics are clamped using screws at the interface of clamping and holding bars and rubber strips are used at the surfaces of geosynthetics layer to prevent any possible slippage. Holder bar is screwed adjustable that allows gripping to move horizontally and provide sufficient resistive tensile force to the geosynthetics layer against possible tensile force arisen during shear testing. The gripping clamp has capacity to grip up to 1 m wide geosynthetic layer.

4.2.2.1(f) Horizontal actuator

To provide shear and tensile forces for interface shear and connection tests respectively, a high capacity double-acting hydraulic cylinder is used as horizontal actuator. The bore diameter of the cylinder used in the apparatus is 180 mm and it is capable of delivering 527 kN (53 ton) push and 423 kN (43 ton) pull forces at 21 MPa (working pressure). Cylinder of 295 mm stroke is used to expedite test setup and to provide sufficient movement to attain peak load of failures for shear and connection tests. A shackle mount is welded at the back of the cylinder to connect with the clevis bracket which is bolted with loading frame. This clevis mount allows the actuator to rotate horizontally against loading frame and facilitates the plunger/piston to align with the setup conditions. Geosynthetic loading clamps is attached with the plunger of 80 mm diameter by clevis joint that assists easy installation of clamping systems.

4.2.2.1(g) Geosynthetic loading clamp

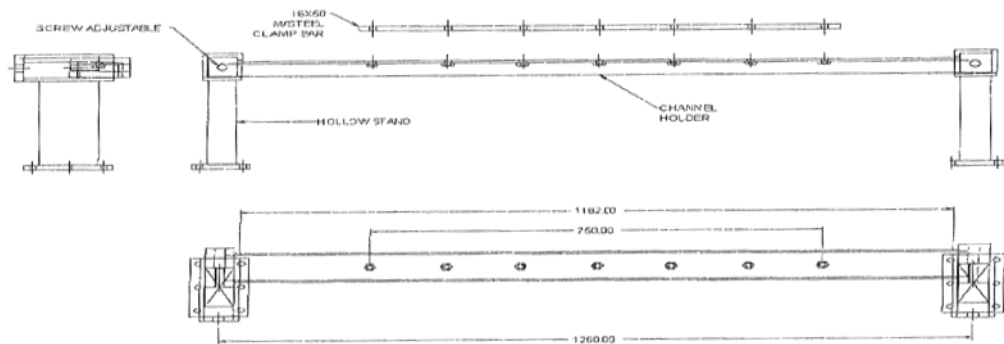
Two different types of loading clamps were designed and used according to the test setups of segmental concrete units. Geosynthetic loading clamp consisting of clamping bars is used as shear loading platen by fixing its clevis joint to provide shear load across the blocks (Figures 4.2 & 4.3(c)). This geosynthetic clamp is capable of applying uniform shear load across top course of 1500 mm in length. It is a two-in-one clamp, used for clamping for stiff geosynthetics like HDPE geogrids (extruded), which are troublesome to roll for gripping in the case of testing. HDPE geogrid is gripped with this clamping system by means of screwing top steel bar with bottom steel bar welded with clamping system (Figure 4.2). Rubber strips are used between the clamping bars for better clamping of extruded geogrids.

For gripping flexible geosynthetics, e.g. polyester geogrid and geotextiles, a roller gripping system was designed and developed. This roller clamping system is more effective for uniform tensile force distribution along the geosynthetic layer. Details of the roller gripping system are shown in Figure 4.4. It is a box type clamping system consisting of two roller bars used for wrapping of geosynthetic reinforcement. The bars are placed freely against trapezoidal steel bars that are bolted with bottom plate of the box. Geosynthetic layer is wrapped through top bar and rolled around bottom bar, and then cover plate is screwed with the trapezoidal bars and tightened enough to keep the rollers together.

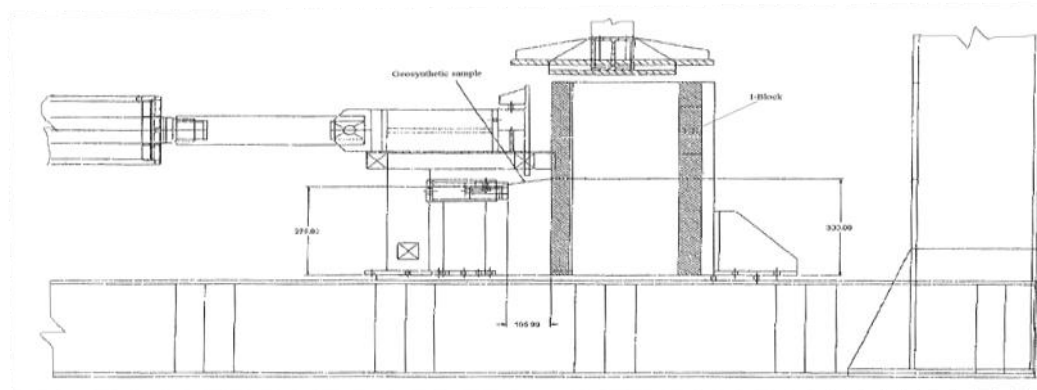
As a result, top roller bar presses the bottom roller bar and hence grips geosynthetics for providing uniform tensile load distribution across the geosynthetic layer. Both of the loading clamps are guided by a support rail, which was designed to minimize friction.



(a) Photograph of gripping clamp

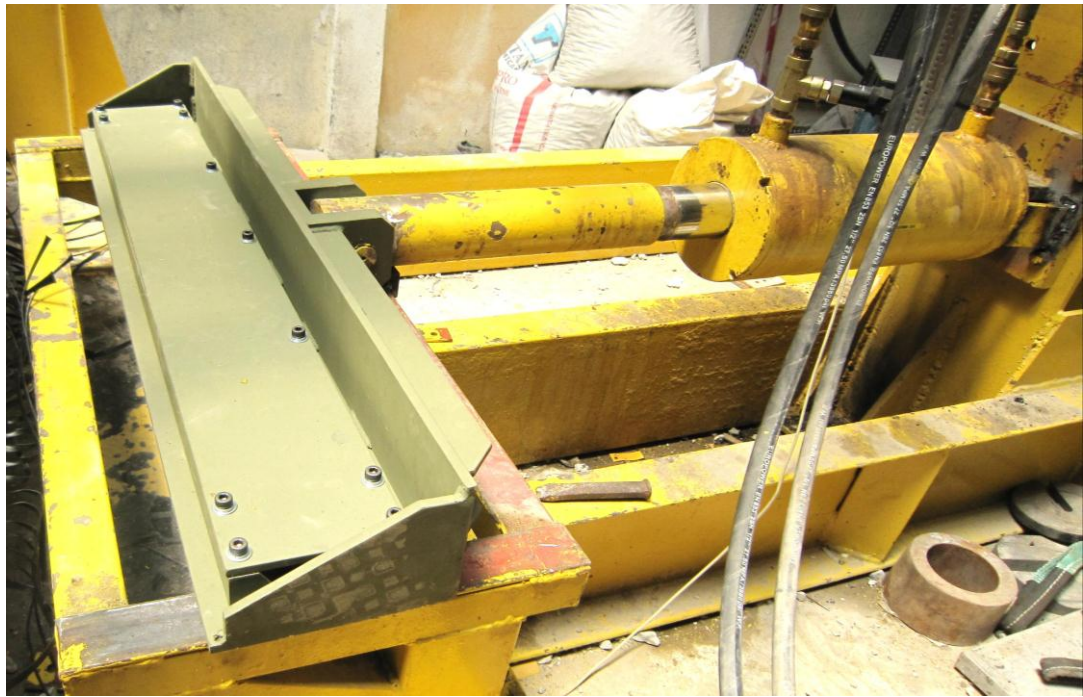


(b) Drawing details of gripping clamp

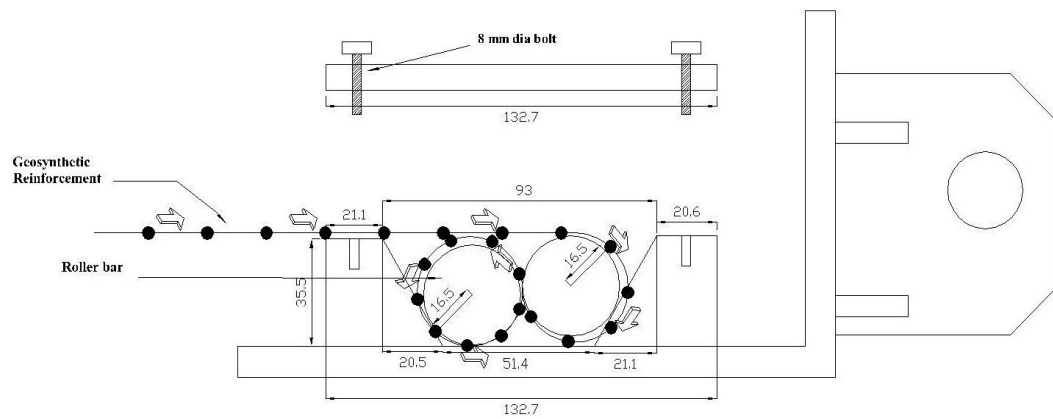


(c) Installation of gripping clamp

Figure 4.3: Details of geosynthetic gripping clamp including photograph, drawing and installation



(a) Photograph of tensile loading clamp



(b) Cross-section of steel roller clamp (courtesy of Soil & Slope Sdn. Bhd.)

Figure 4.4: Details of geosynthetic loading clamp

4.2.2.2 Electric pump system

The electric pump system was fabricated locally using available hydraulic accessories in Malaysia. Pumps are mechanical devices used to move fluid by suction or pressure. Two gear pumps of different displacement capacities are selected for two hydraulic jacks. Small pump (rear) of $0.98 \text{ cm}^3/\text{rev}$ displacement capacity is used for vertical actuator with its working pressure up to 23 MPa (recommended). For horizontal actuator, a relatively big pump (front) of $6.55 \text{ cm}^3/\text{rev}$ displacement capacity is used with its recommended working pressure is 25 MPa. Two pumps are combined with each other according to manufacturer's design and then connected with the shaft of an induction motor of 2.2 kW capacity. Pumps and motor are installed over the reservoir tank, which is filled up using high viscous hydraulic oil. High pressure (27 MPa) hoses made of synthetic nitrile rubber liner and reinforced by two braids of high tensile steel wire are chosen for hydraulic systems. Hoses transport high viscous pressurized fluid in whole hydraulic circuit. Two main parts of the apparatus; actuators and pump system are linked each other by means of four hoses. The ends of the hoses are connected with the cylinders and pump system using couplers (male- female) and manifolds. The electric pump system can easily be dismantled from the cylinders by unplugging the male and female couplers and therefore the pump system can be set in any convenient place according to the apparatus installment.

Two 4-way directional control valves with pressure adjustable knob are mounted in the pump system with a view to controlling the direction of hydraulic fluid easily in the double-acting system (cylinders). The directional valves for the vertical and horizontal actuators are operated manually using lever arm. To monitor the pressure reading of the hydraulic system two pressure gauges are also attached with the advance ports of manifold (Figure 4.5).

Flow rate of horizontal cylinder was regulated by a flow restrictor adjustable (flow rate 15000 cm³/min) which was installed initially on the fluid line of horizontal actuator to control its linear displacement (Figure 4.1). But during the sample shear testing, it was observed that this flow adjustable valve is not able to control linear displacement. This had been recommended by the available protocols used for full scale laboratory study of SRW units. Therefore, a new flow regulator valve of controlling maximum regulated flow 1500 cm³/min was attached to control recommended displacement 1 mm/min and 20 mm/min for shear and connection test, respectively. This valve has pressure compensator which makes the controlled flow independent of pressure variation and it can be used up to 21 MPa working pressure. Another flow controlled valve (like horizontal one) of regulated flow 6000 cm³/min (max.) was used to control the plunger movement of the vertical actuator because of speedy movement of plunger of the vertical actuator. It was noticed that speedy movement or drop of normal loading platen attached with plunger causes failure of top blocks. This may be happened due to the gravity force acting on heavy weight vertical loading platen. The vertical flow control valve was mainly installed to apply normal load on the blocks at a nominal speed as well as fast restoration of the cylinder. Figure 4.5 illustrates pressure and flow controlling system of the pump. Details of hydraulic system of the electric pump are sketched in Figure 4.6.

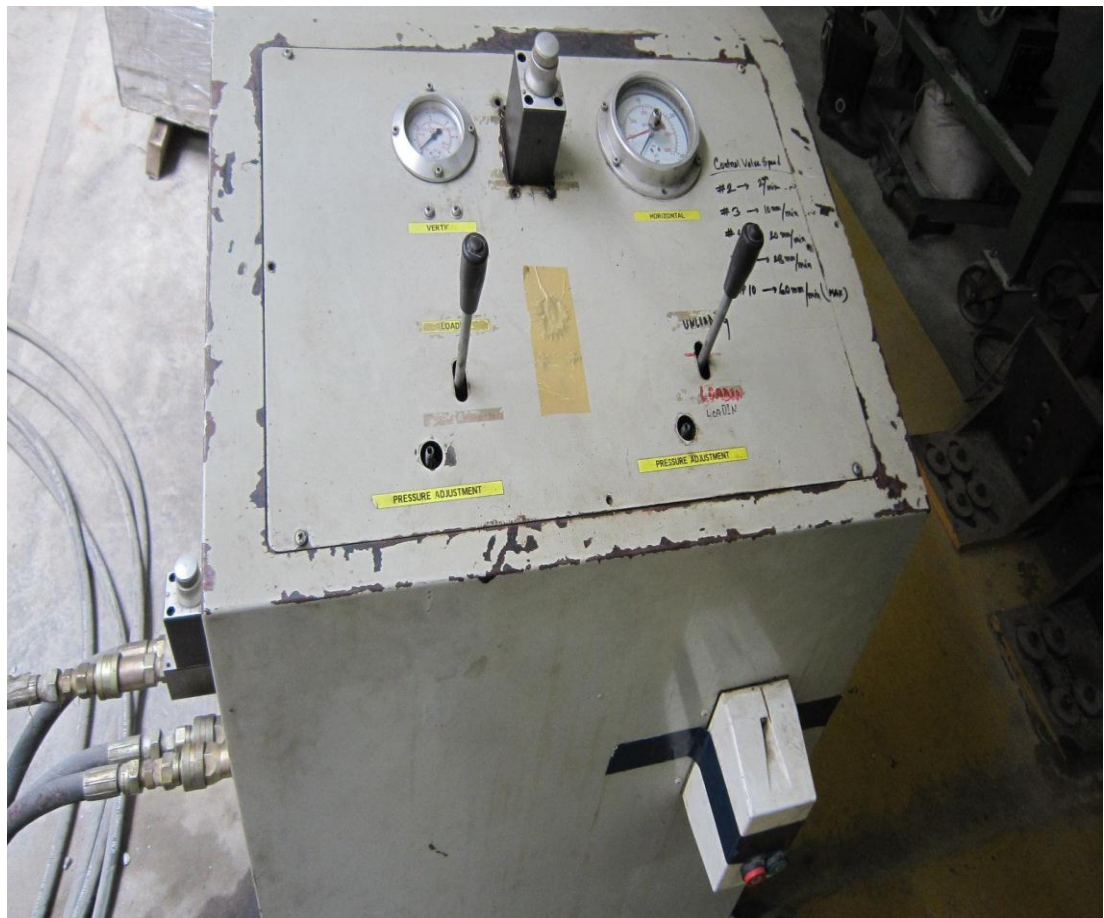


Figure 4.5: Electric pump system

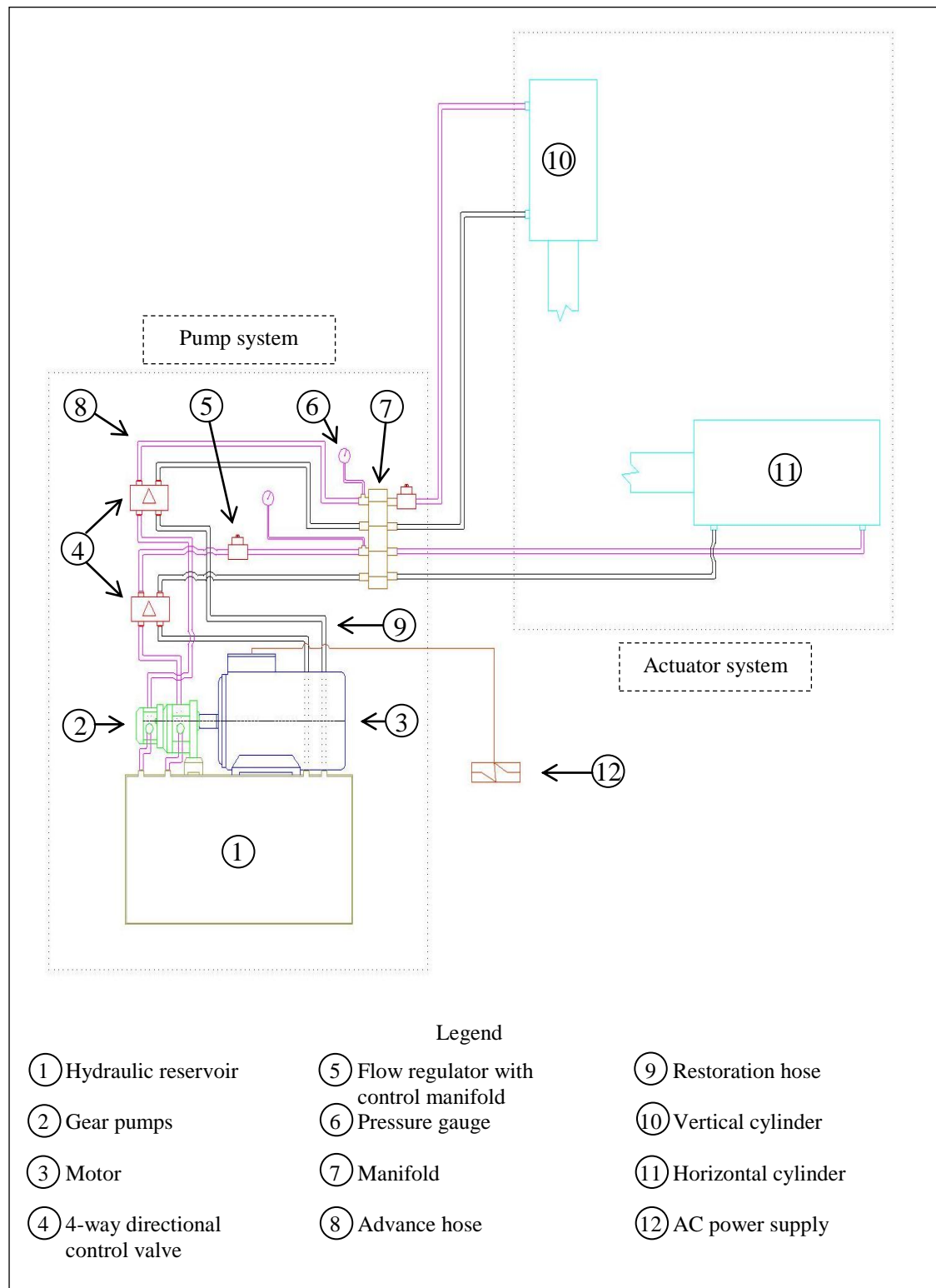


Figure 4.6: Hydraulic circuit of pump

4.2.3 Instrumentation and data acquisition

The rate of displacement (mm/min) of horizontal actuator is calibrated against flow control valve using linear variable displacement transducers (LVDTs). Displacement transducers of 50 mm capacity are also used to monitor shear displacement of during interface shear testing. To get precise pressure reading from the actuators, two pressure transducers of 25 MPa capacities are mounted with the actuators.

A high capacity tension/compression load cell is used to calibrate the cylinders against pressure transducers. During testing, all measurements are recorded at particular time interval in a high resolution data logger.

4.2.4 Performance of surcharge loading arrangement

In the modified test apparatus, a moveable vertical actuator is mounted with loading frame after considering the effects of fixed vertical actuator. Bathurst et al. (2008) reported that with fixed vertical actuator/piston arrangement increases normal load with shear displacement rather than becoming constant. It is occurred due to bending of vertical piston that causes locking of the piston with top block during shear and hence increase normal load (Figure 4.7). As the sample test, Figure 4.8 evaluates performance of moveable vertical cylinder against shear displacement. From the Figure 4.8, it is clearly seen that normal load variation is almost constant over the period of shear testing although there is very little fluctuation that can be ignored easily. It is resulted due to the presence of steel rollers in between the vertical piston and loading frame (Figure 4.2), which allow the piston to move horizontally without any bending against shear displacement and keep normal load constant throughout interface shear testing.

Figure 4.8 outlines the interface shear behavior of I-Block infilled with granular materials under an average normal stress of 160 kPa. The details about the interface shear testing are outlined in the following section.

4.2.5 Advantages of the modified apparatus

The competitive advantages of the modified apparatus can be summed up as follows:

1. Moveable vertical loading assembly provides constant surcharge load with respect to fixed vertical actuator. It is also uncomplicated and time-saving testing arrangement regarding to airbag arrangement recommended by available test protocols.
2. It is a well-suited device for full scale laboratory study for all types of facia units (SCUs). The apparatus can be dismantled and adjusted according to the block geometry and test setup.
3. A newly designed roller gripping and loading clamp provides better gripping and tensile force distribution along geosynthetic layer.
4. The apparatus offers a wide range of displacement speed (1mm/min-60 mm/min) for horizontal actuator.
5. The apparatus can easily be used for full scale laboratory study of relatively high and long wall system because of its capacity and loading assembly.

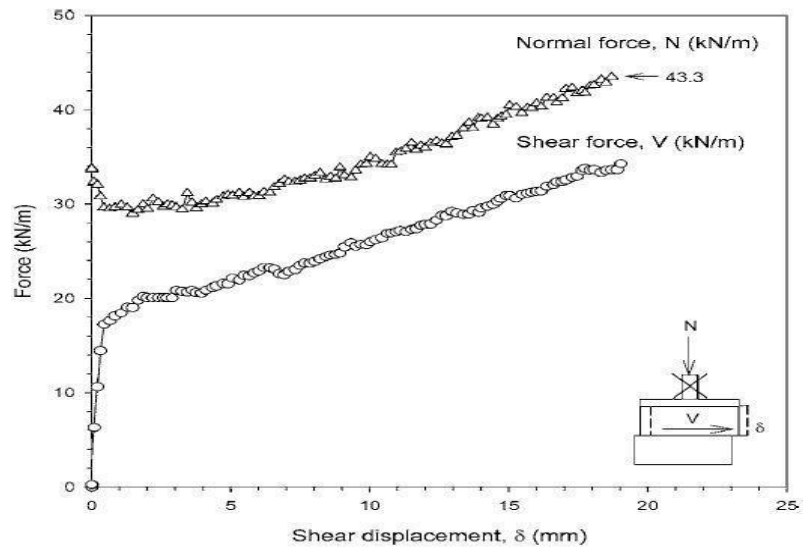


Figure 4.7: Normal load response against shear displacement for fixed vertical loading arrangement (Bathurst et al. 2008)

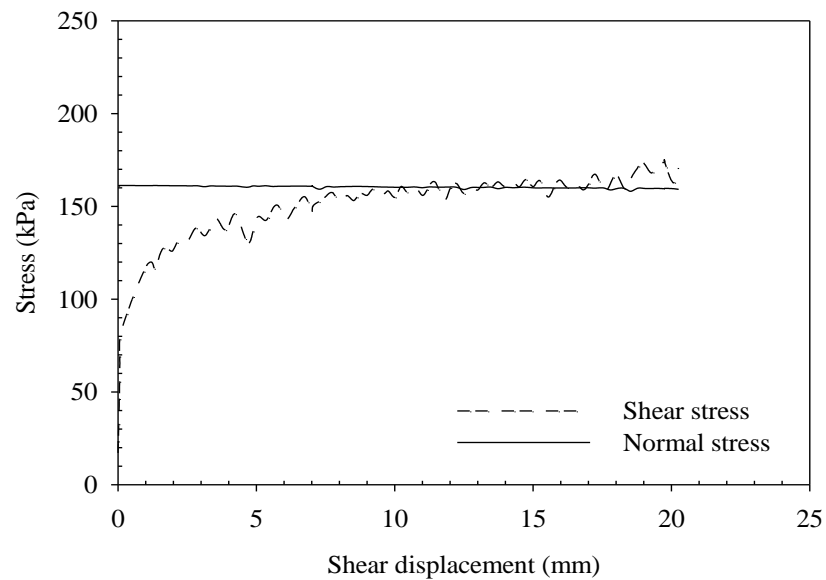


Figure 4.8: Normal load response against shear displacement for moveable vertical loading arrangement

4.3 Test arrangement and procedure

On basis of the research objectives, the testing program of interface shear tests was divided into three groups. These tripartite groups were made to find out the effects of different interlocking materials (pins and granular in-fills) and inclusions used in this research investigation. An outline of the test groups are given in Table 4.1. A general description of interface shear tests for I-Blocks is outlined in the following section.

4.3.1 Interface shear tests

A general test setup for interface shear tests with I-Block system is illustrated in Figure 4.9. According to the test protocols (NCMA SRWU-2, ASTM D 6916-03), two layers/courses of modular block units were used for interface shear tests. The bottom course consisting of two I-Blocks was placed on platform to coincide running joint with the centerline of the horizontal actuator and braced laterally against restraining plate. The back of bottom course was fixed by using a back support beam, which was bolted with platform to stop bending of bottom course during shear testing.

In the case of granular in-fills, the hollow space between the blocks was filled up with aggregates and lightly compacted using a steel rod. Due to tapered rear flange, a small steel anchored plate was placed at back of bottom course to fill up the gap in between two blocks and to hold compacted aggregates. Depending on the test conditions, one end of the geosynthetic sample was placed over the bottom course and connected with the shear pins. The other end of geosynthetic layer was gripped to the steel clamp for preventing any possible slippage of the reinforcement layer during shear testing. Geosynthetic layers were trimmed according to the block's interface and gripping system.

A single I-Block was placed (with zero setback) centrally over the running joint formed by the two underlying units to simulate the staggered construction procedure used in the field. The double open-ended space of the top block was filled up with aggregates and two (2) steel plates were used to hold the infilled aggregates of the top block.

Surcharge/Normal load was imposed by vertical actuator only over the top block through stiff rubber mat and simulated an equivalent height of stacked blocks. The shear/horizontal load was applied against the top course and immediately above the shear interface to minimize moment loading at a constant rate of 1 mm/min of horizontal actuator (ASTM D 6916-03). A steel plate with a gum stiff rubber mat was attached to geosynthetic loading clamp (Figure 4.9) to concentrate shearing load only over the centrally installed top block. A horizontal seating load of 0.22kN was applied to the top block to ensure close fitting of the block systems and after that the load and displacement devices were set to zero (NCMA, SRWU-2).

The shear displacement and load/pressure reading were continuously measured and recorded during the tests by a data logger. The data were recorded at every 10 second interval.

Tests were continued until failure of shear resistance occurred. To check the accuracy of the test executions, three identical tests were performed at different normal loading conditions. Test results were characterized under two criteria: peak (ultimate) shear strength at failure and service state shear strength at 6 mm displacement (2% of I-Block height), which is recommended by Collin (1997) in NCMA design guideline.

For each test, new shear pins and geosynthetic reinforcement were used. As usually, the blocks used in the tests were new and free from any visual cracks. In order to minimize the use of new blocks for repeated tests, first time tested/used blocks (free from any damage) were reused by interchanging their positions to provide undamaged interface for subsequent testing. The used blocks were interchanged according to clockwise direction started from top block.

Table 4.1: Shear test combinations for different interface conditions

Group	Configuration	Infill	Shear pin	Inclusion
1	Type 1	N/A	N/A	N/A
"	Type 2	"	Steel	"
"	Type 3	"	Plastic	"
2	Type 4	NCA	N/A	N/A
"	Type 5	RCA 1	"	"
"	Type 6	RCA 2	"	"
"	Type 7	NCA	Steel	"
"	Type 8	NCA	Plastic	"
3	Type 9	NCA	Plastic	Geogrid 1
"	Type 10	RCA 1	"	"
"	Type 11	RCA 2	"	"
"	Type 12	NCA	"	Geogrid 2
"	Type 13	RCA 1	"	"
"	Type 14	RCA 2	"	"
"	Type 15	NCA	"	Geotextile
"	Type 16	RCA 1	"	"
"	Type 17	RCA 2	"	"

Note: Configuration refers interface condition; N/A = not applicable

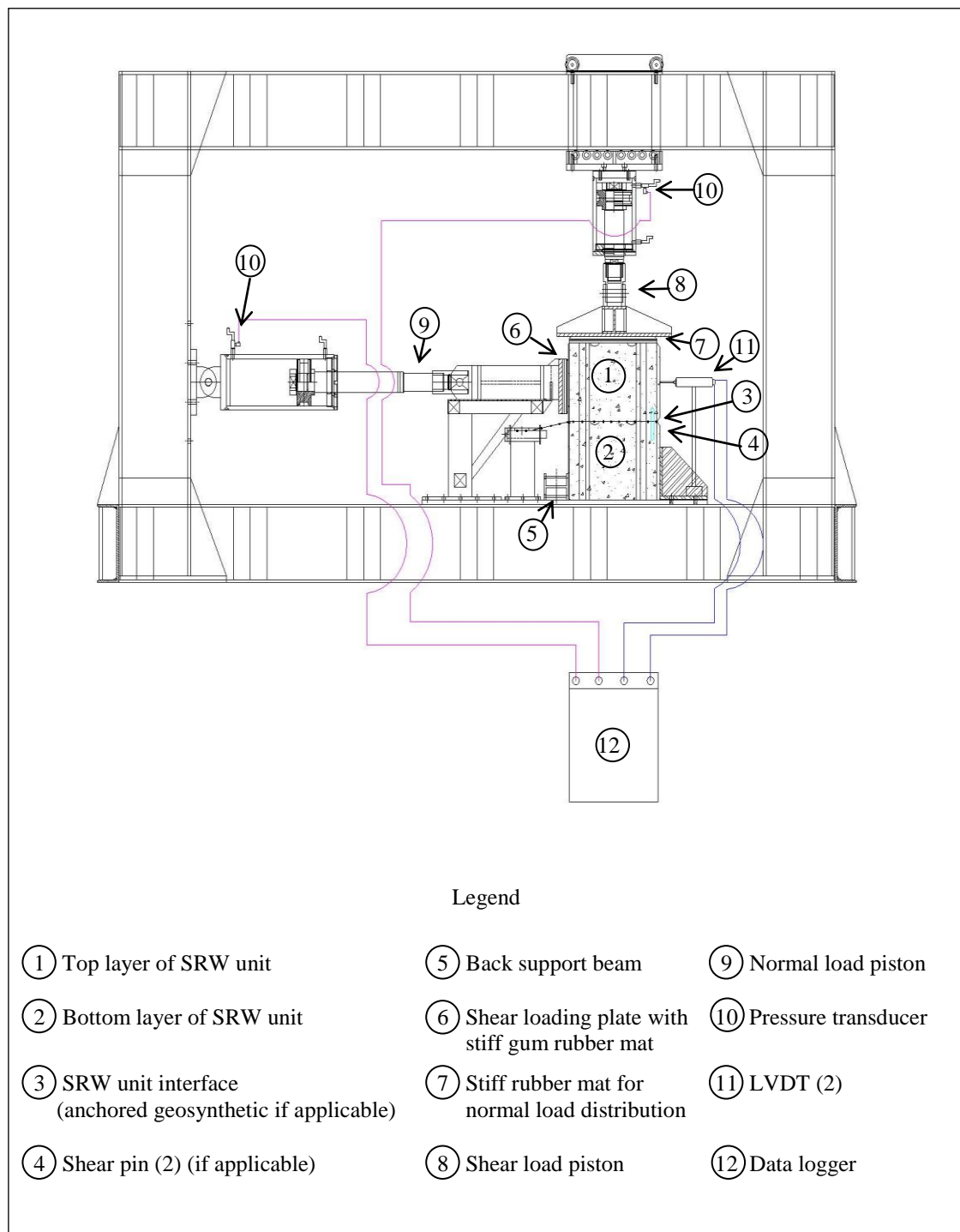


Figure 4.9: Generic interface shear testing arrangement

4.3.2 Calculations

For each normal load level, shear stress-displacement relationship was plotted to compare the frictional behavior of I-Block system under different interlocking and inclusion conditions at interface. Shear stress under peak (ultimate) and service state were calculated using equations 4.1 and 4.2 as follows:

$$\text{Ultimate shear stress, } V_p = F_p/A_i \quad (4.1)$$

$$\text{Service state shear stress, } V_{ss} = F_{ss}/A_i \quad (4.2)$$

Where:

V_p = Ultimate (peak) shear stress per length of top block (kPa)

V_{ss} = Service state shear stress at 6 mm deformation (kPa)

F_p = Ultimate (Peak) shearing load (kN)

F_{ss} = Measured shear load at 6 mm deformation (kN)

A_i = Total area of the interface surface (m²)

In this research, Mohr-Coulomb failure criterion was used to find out angle of friction (λ) and apparent cohesion (α) for each group of tests. Performance parameters (α & λ) for I-Block systems at ultimate and service state strength criteria were evaluated using equations 4.3 and 4.4 as follows:

$$\text{Ultimate shear stress, } V_p = N \tan \lambda + \alpha \quad (4.3)$$

$$\text{Service state shear stress, } V_{ss} = N \tan \lambda' + \alpha' \quad (4.4)$$

Where:

V_p = Peak shear capacity (kPa)

V_{ss} = Service state shear capacity (kPa)

N = Normal stress (kPa) at block interface

λ = Peak angle of friction (degrees)

λ' = Service state angle of friction (degrees)

α = Peak apparent cohesion (kPa)

α' = Service state apparent cohesion (kPa)

4.3.3 Details of test groups

4.3.3.1 Group 1 (Effect of rigidity of shear connector)

The underlying aim of this group of tests was to examine the effect of shear pin rigidity on interface shear capacity. To compare the effects of mechanical connectors on interface shear behavior of modular block units, two types of shear pins (steel & plastic) were selected. Steel pins are normally used in segmental wall system to help out facing alignment. By considering the rigidity of steel pin, relatively flexible plastic made of UHMWPE was applied in this investigation. The cavities of the blocks were not filled with gravel to minimize the number of parameters to avoid its influence on the test results. Geosynthetic inclusions were also not used at the block interface because it may influence on interface shear capacity. The influence of rigidity of shear pins on interface shear capacity was compared against purely frictional behavior. Figure 4.9 shows the photograph of typical test setup for Group 1.

4.3.3.2 Group 2 (Effect of recycled coarse aggregate as in-fillers)

The main objective of this group of tests was to examine the effect of recycled coarse aggregates on interface shear capacity. As granular in-fills, two types of recycled aggregates were used along with natural aggregates. Recycled aggregates were mainly selected based on the compressive strength of the source waste concretes to investigate the effect of strength property on frictional behavior of recycled aggregates used as in-fillers. Purely frictional capacity of I-Block infilled with recycled aggregates was compared to against those with infilled by fresh aggregates. Pins were not used in purely frictional shear to minimize its effect on interface shear capacity of I-Block system infilled with gravels.

To simulate the actual field condition of I-Block wall, combined interlocking (gravel & pins) effect on interface shear capacity was also investigated. Figure 4.10 illustrates the photograph of typical test setup for Group 2.

4.3.3.3 Group 3 (Effect of geosynthetic inclusion)

The main objective of this group of tests was to examine the effect of geosynthetic inclusion on interface shear capacity and frictional performance of geosynthetic reinforcement with recycled aggregates used as granular in-fills in this investigation. In this investigation, three types of geosynthetic reinforcements were chosen: a flexible PET-geogrid (#1), a stiff HDPE-geogrid (#2), and a flexible PET-geotextile which are mostly used in Malaysia for GR-SRW constructions. As shear pins, plastic pins were used because of its better performance over last two groups of tests. To observe the actual field condition of I-Block wall with geosynthetic inclusion, the hollow spaces of the blocks were filled up with all types of aggregates used in this research.

Frictional performance of the I-Block systems infilled with aggregates were compared to different geosynthetic inclusion condition. Figure 4.11 demonstrates the photograph of typical test setup for Group 3.

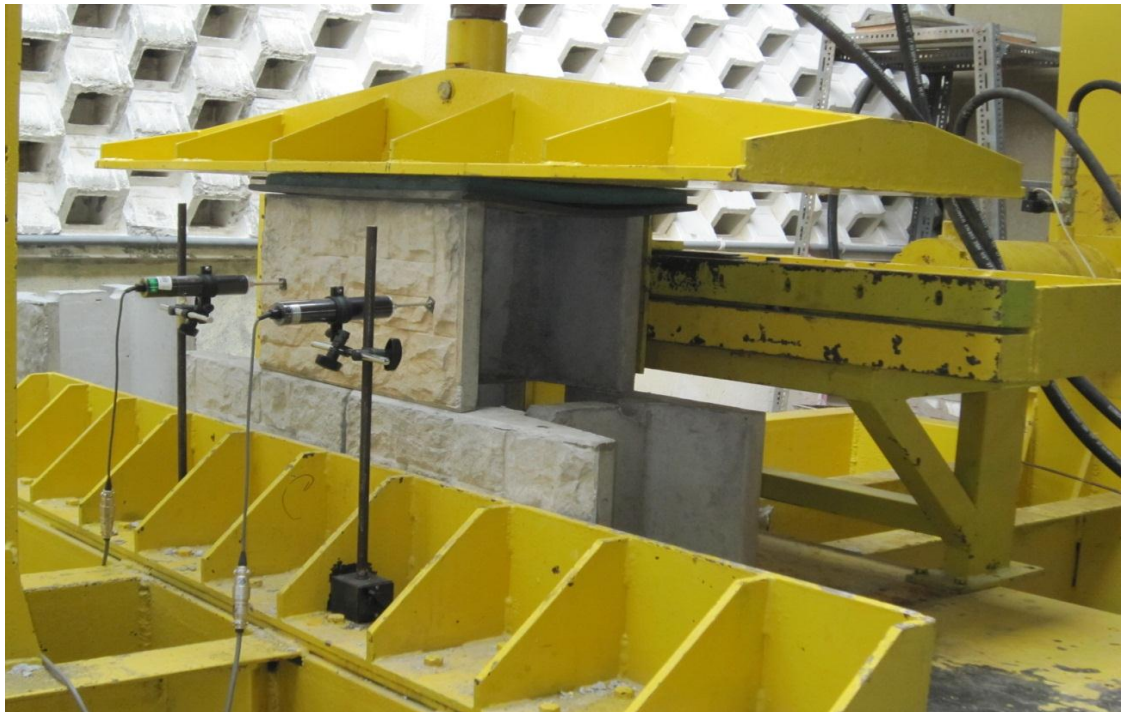


Figure 4.10: Photograph of typical test setup for Group 1 showing rubber mat and LVDTs



Figure 4.11: Photograph of typical test setup for Group 2 showing rubber mat steel plate, and LVDTs



Figure 4.12: Photograph of typical test setup for Group 3 showing geotextile sample and gripping system

CHAPTER 5 TEST RESULTS AND COMPARISON

5.1 General

This Chapter presents the experimental results of all groups' tests as detailed in Chapter 4. The data are outlined in sections according to test Groups 1, 2 and 3. This chapter also compares the results of different configurations in each group. The explanation and discussion of results are provided in Chapter 6.

5.2 Group 1: Effect of rigidity (stiffness) of shear pins on interface shear capacity

5.2.1 Overview

Group 1 was divided into 3 configurations of tests series e.g. Types 1, 2 and 3. The main variable among the test series was stiffness of shear pins. Stiffness of the shear pins varied from zero (no shear pins which allow block to move freely) for referenced (control) configuration Type 1 to very high (steel pins) for Type 2. Another configuration (Type 3) was selected for a medium stiffness of shear pins (plastic pins) falling between the limiting stiffness cases (zero to very high). Frictional performance of hollow I-Block system was examined under three different normal load conditions. Infill materials were not used to minimize the effect of other parameters on concrete to concrete friction that could influence the test results.

5.2.2 Type 1 (Concrete-to-concrete interface)

In this configuration, the interface shear behavior of empty I-Block system was investigated without shear pins. Figure 5.1 illustrates the frictional performance of empty I-Block system under different surcharge pressures.

Hollow I-Block system fails beyond 120 kPa during interface shear testing although the net compressive strength of it is 8000 kPa (Figure A.1). This may be happened due to stress concentration at flanges' contact area. For brevity and better presentation, only selected results of repeated tests are presented here.

The shear stress-displacement curves follow the basic frictional behavior e.g. shear resistance is proportional to the applied normal load. The shear displacement was calculated as the average of relative displacements of top block against bottom layer measured by two LVDTs situated at the edges of top block. Nominally identical curves demonstrate the accuracy of the performed laboratory tests. The peak shear stresses of these tests are less than $\pm 10\%$ from the mean of the repeated tests. The vertical dashed line in Figure 5.1 reports the serviceability limit, which is around 6 mm according to the block geometry (2% of the I-Block height). The serviceability criterion is determined to compare the ultimate (peak) shear capacity of modular block system with service state capacity. Figure 5.2 shows the interface shear capacity of empty I-Block system under ultimate (peak) and service state (deformation) criteria. To get best fit lines for shear capacity envelopes, all repeated test data are considered to represent the results of repeated tests with those which peak shear stresses are less than $\pm 10\%$ from the mean of the tests. The data of Type 1 are used as reference data that compares the effect of shear connectors on interface shear strength.

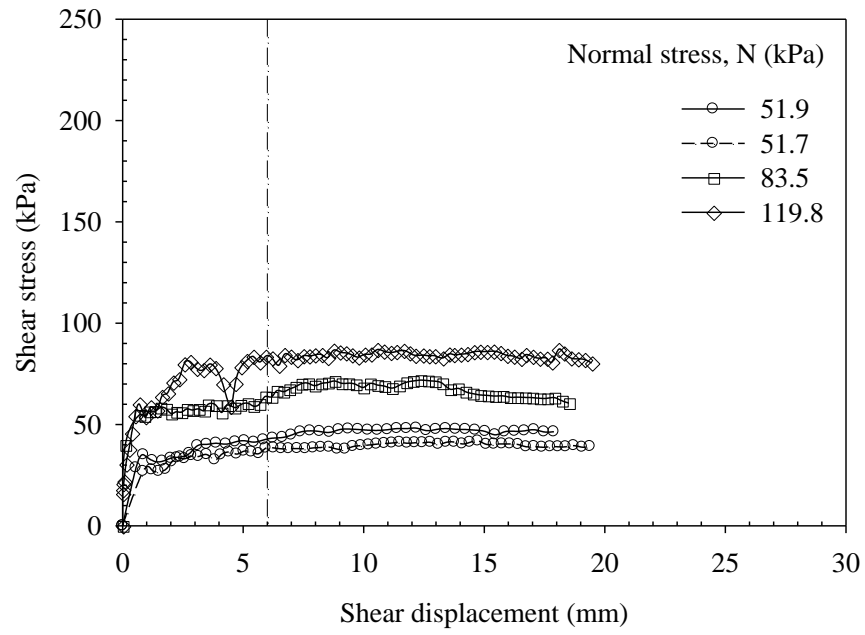


Figure 5.1: Shear stress versus displacement for Type 1 (hollow facing unit)

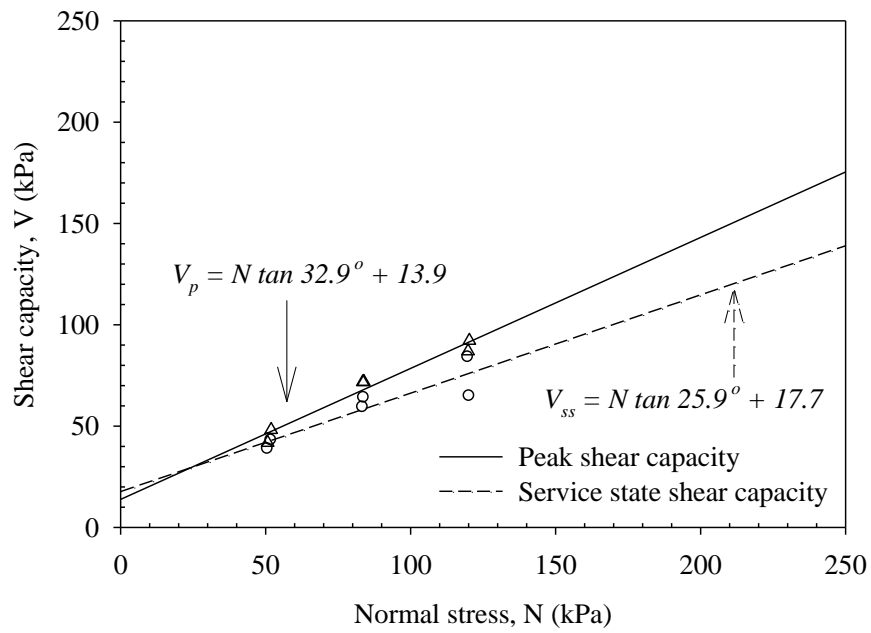


Figure 5.2: Interface shear capacity versus normal stress for Type 1 (hollow facing unit)

5.2.3 Type 2 (Concrete-to-concrete interface with steel shear pins)

The objective of this configuration was to identify the effect of steel shear pins on interface shear capacity. Figure 5.3 demonstrates the frictional performance of empty I-Blocks with steel pins (high stiffness) against shear displacement at different normal loading conditions. It also outlines the three identical tests at a normal stress of about 50 kPa. The shear stress-displacement curves demonstrate typical saw-tooth patterns of shear stresses with displacements that result from the stress concentrations at joints due the presence of steel shear pins leading to the failure of blocks before survivability limit (6 mm). Initial (peak) capacity for the I-Block system with steel pins is shown in Figure 5.4.

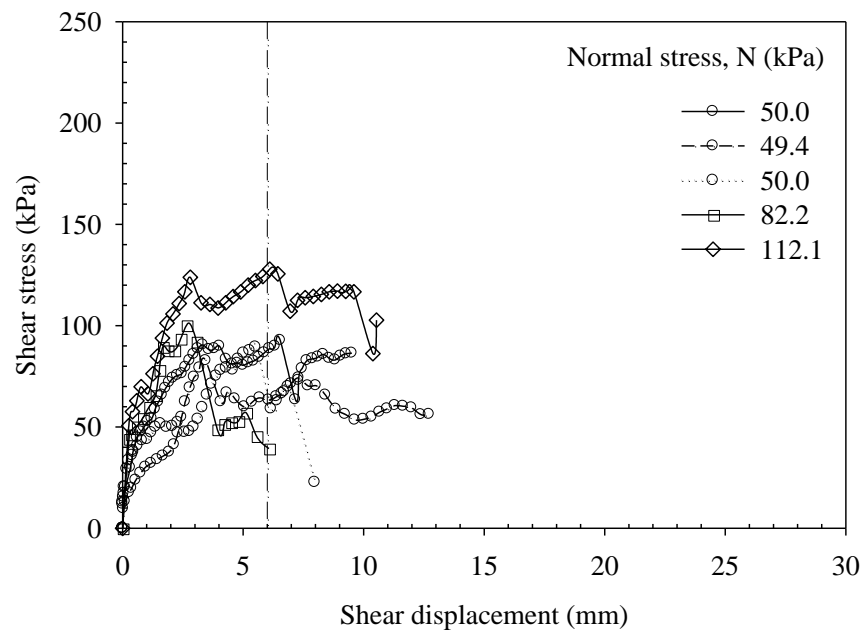


Figure 5.3: Shear stress versus displacement for Type 2 (hollow facing unit with steel pins)

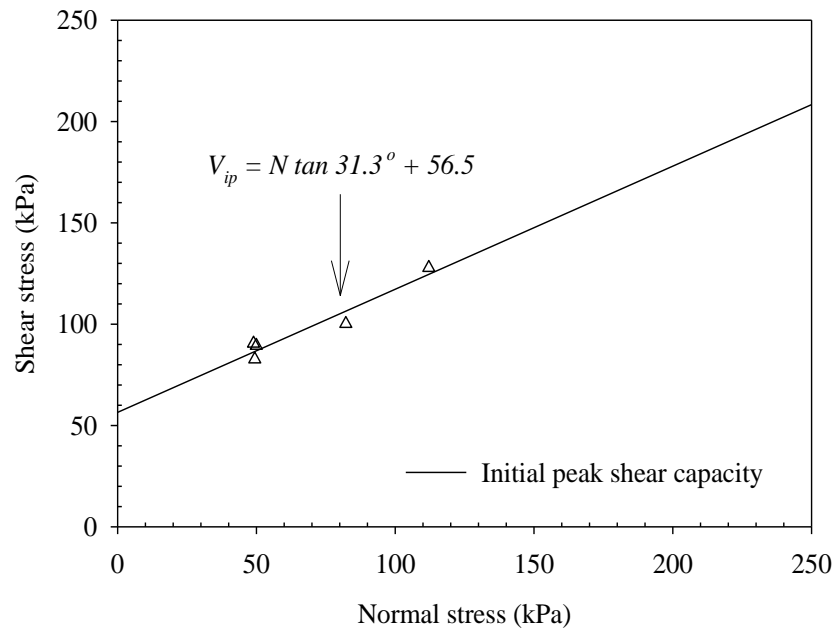


Figure 5.4: Interface shear capacity versus normal stress for Type 2 (hollow facing unit with steel pins)

5.2.4 Type 3 (Concrete-to-concrete interface with plastic shear pins)

The results of Type 3 configuration report the influence of plastic on interface friction behavior of the tested segmental (modular) block system. The curves of Figure 5.5 view magnitude and distribution of shear stress for segmental concrete units (I-Blocks) against shear displacement with plastic pins used as shear pins. From the Figure 5.5, it is seen that shear stress increases quickly at the beginning and after certain displacement drops gradually. The maximum shear stresses of the repeated tests for a normal load of about 53 kPa slightly varied due to the effect of clear shear of both pins installed at the connection joints. Shear stress against normal stress data are plotted in Figure 5.6 to compare the shear capacity envelopes under peak and deformation (serviceability) criteria. From the Figure 5.6, it is found that angle of friction under serviceability condition is higher than peak condition.

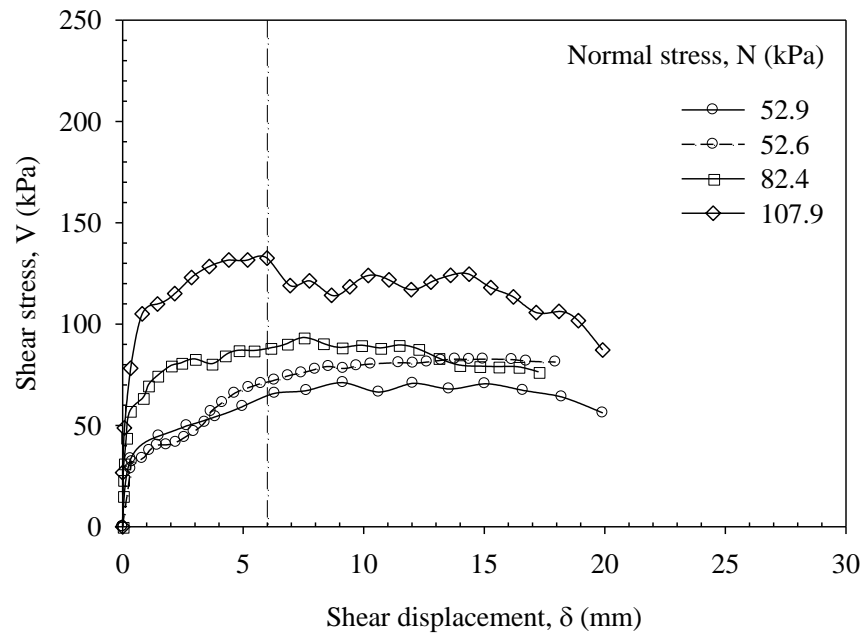


Figure 5.5: Shear stress versus displacement for Type 3 (hollow facing unit with plastic pins)

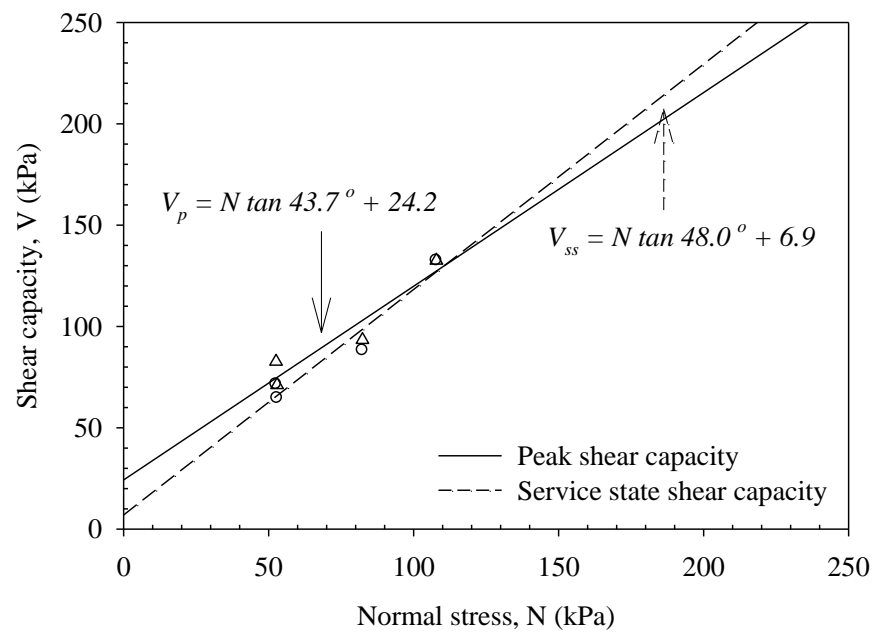


Figure 5.6: Interface shear capacity versus normal stress for Type 3 (hollow facing unit with plastic pins)

5.3 Group 2: Effect of recycled aggregates (granular in-fills) on interface shear strength

5.3.1 Overview

Group 2 consists of five configurations of test series which are Types 4, 5, 6, 7 and 8. The effects of recycled aggregates along with natural aggregate as granular in-fills were investigated in the first three configurations (Types 4, 5 and 6). Natural coarse aggregate (NCA) was used for Type 4 and recycled coarse aggregates (RCA 1 and RCA 2) were used for Type 5 and 6 respectively. In these configurations, mechanical connectors (shear pins) were not used to examine the influence of recycled aggregates against natural (fresh) aggregates on interface shear strength of infilled block system. In last two configurations (Type 7 and 8); series of tests were executed to identify the frictional performance of infilled block system with shear pins used in Group 1. Type 7 and 8 investigates the shear capacity of infilled I-Block system with steel and plastic pins respectively. As a granular infill for Type 7 and 8, natural coarse aggregate (NCA) was used. Type 4 is a referenced configuration for this Group.

5.3.2 Type 4 (Concrete-to-concrete interface with granular infill, NCA)

Effect of granular infill on interface shear capacity of hollow block system was investigated by this series of tests. In this configuration, natural (fresh) coarse aggregate was used as infill material. Figure 5.7 shows the interface shear strength of I-Block system infilled with natural aggregates under different normal loading conditions. From the Figure 5.7, it is viewed that the shear stress for each normal stress increases gradually and eventually reached the maximum value after a significant amount of displacement. Figure 5.7 also illustrates three repeated tests under a normal stress of about 124 kPa to justify the accuracy of the performance (laboratory) tests.

The interface shear capacity envelopes of the I-Block system infilled with NCA are demonstrated in Figure 5.8. It also outlines the performance parameters (friction angle and apparent cohesion) of infilled I-Block system. Serviceability envelope goes through the origin and almost parallel to the peak shear capacity envelope. The data of Type 4 are considered as the referenced (control) data to compare the effect of other infillers used in this study for facing units.

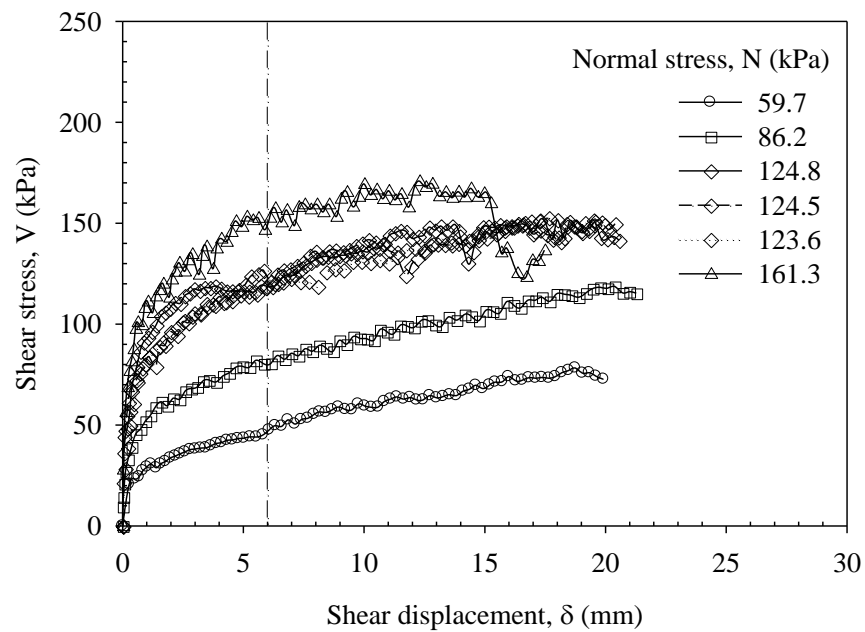


Figure 5.7: Shear stress versus displacement for Type 4 (hollow facing unit infilled with NCA)

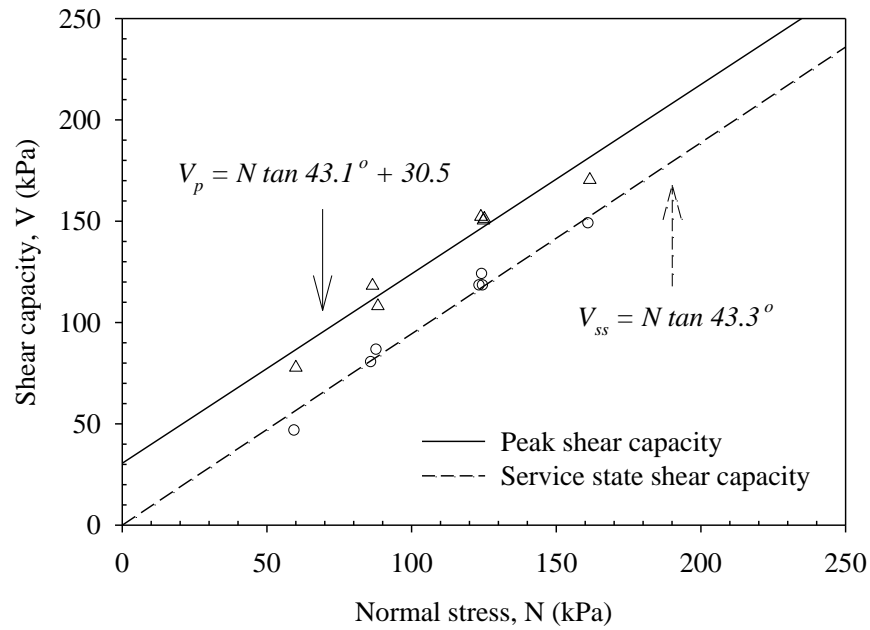


Figure 5.8: Interface shear capacity versus normal stress for Type 4 (hollow facing unit infilled with NCA)

5.3.3 Type 5 (Concrete-to-concrete interface with granular infill, RCA 1)

In comparison to NCA, recycled coarse aggregate (RCA 1) was used as granular infill in this configuration. Frictional behavior of hollow segmental retaining wall units infilled with RCA 1 is outlined in Figure 5.9. The shear stress-displacement curves of Figure 5.9 illustrate almost similar distinctive pattern of shear stress increment as described in Figure 5.7. The shear capacities (peak and service state) for infilled blocks with RCA 1 are given in Figure 5.10.

From the Figure 5.10, it can be seen that service state shear capacity totally depends on angle of friction when its apparent cohesion (normal load independent shear strength) is zero.

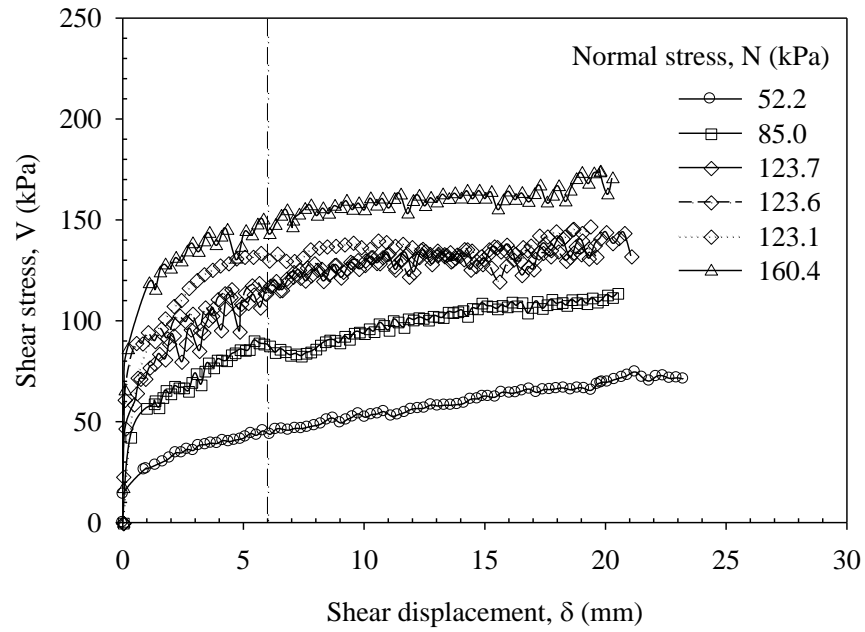


Figure 5.9: Shear stress versus displacement for Type 5 (hollow facing unit infilled with RCA 1)

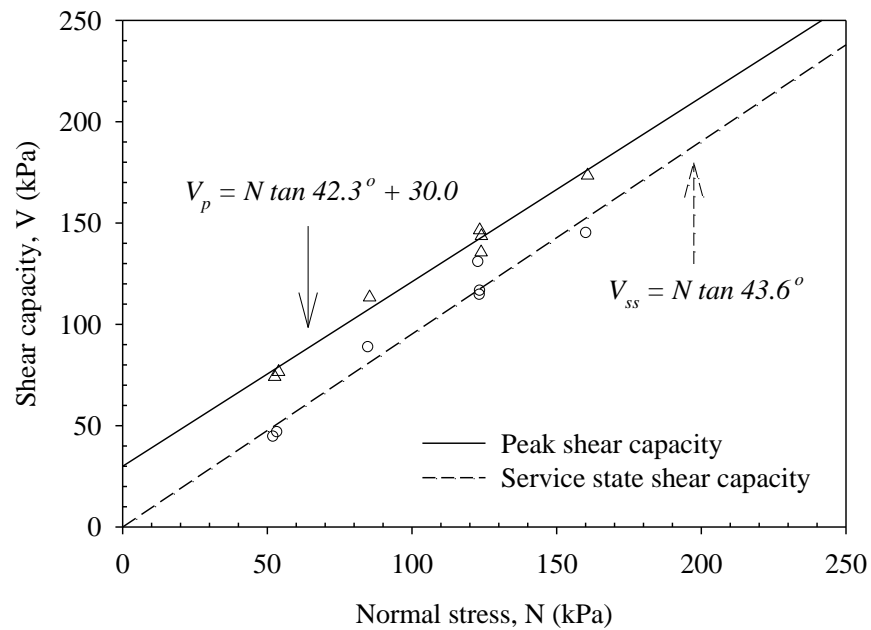


Figure 5.10: Interface shear capacity versus normal stress for Type 5 (hollow facing unit infilled with RCA 1)

5.3.4 Type 6 (Concrete-to-concrete interface with granular infill, RCA 2)

In this test series, a recycled aggregate produced from high strength waste concrete was used. To find out the effect of granular infill (RCA 2), a series of tests were executed under different surcharge levels. Shear stress-displacement curves for each normal load are shown in Figure 5.11. It is seen that shear stress-displacement curves for normal stress of about 123 kPa is more wavy, which may be happened due to the stress concentration at concrete-to-concrete interface because of irregularity of block's surface.

On the other hand, Figure 5.12 elucidates the shear capacity envelopes for this configuration. From the Figure 5.12, it is seen that the performance parameters (friction angle and apparent cohesion) under serviceability criterion are lower than peak criterion.

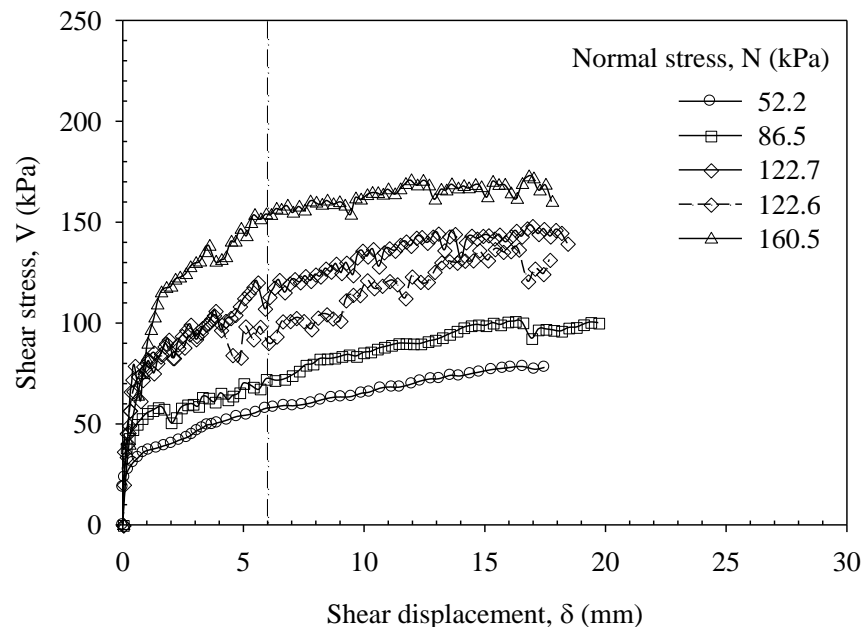


Figure 5.11: Shear stress versus displacement for Type 6 (hollow facing unit infilled with RCA 2)

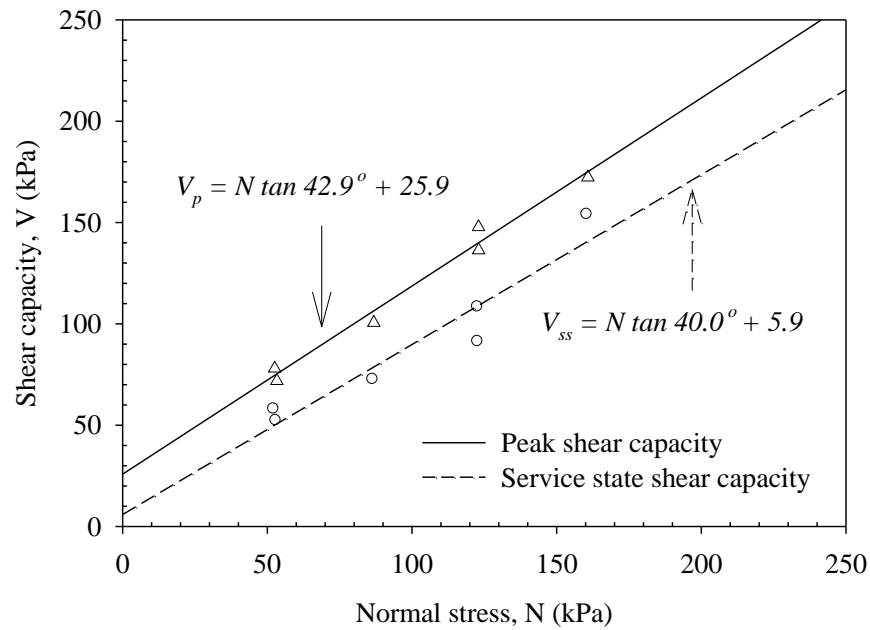


Figure 5.12: Interface shear capacity versus normal stress for Type 6 (hollow facing unit infilled with RCA 2)

5.3.5 Type 7 (Concrete-to-concrete interface with steel pin and NCA)

This combination of tests evaluates the frictional performance of infilled block system with steel shear pins used as mechanical connectors for facing alignment. In this series, NCA was selected as an in-filler to represent the frequent used granular material in retaining wall constructions. Figure 5.13 demonstrates shear stress-displacement curves under different normal loading levels. The curves of Figure 5.13 show peak values before and after serviceability limit for all surcharge pressures. As usually, peak of shear stress occurs at low shear displacement (< 6 mm) after which shear stress decreases to a lower maximum stress. But the curves for a normal stress of about 86 kPa show the peaks after a sufficient amount of displacement (> 6 mm) and then heading towards a lower minimum value. The shear capacity envelopes for this series of test are shown in Figure 5.14. The data presented in Figure 5.14 illustrates the shear capacities under initial peak and a displacement of 15 mm conditions. It was observed during shear testing that block fails at connection joints after initial peak (< 6 mm).

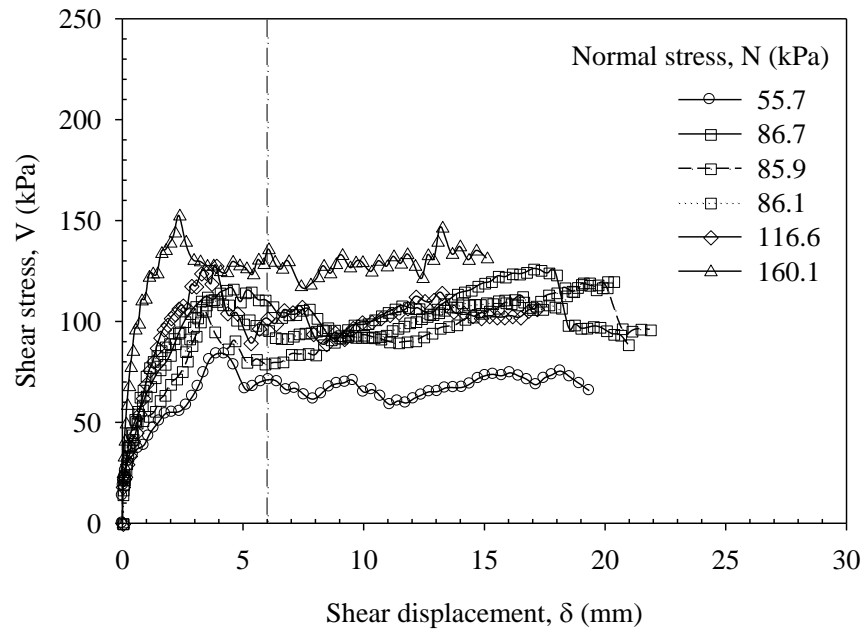


Figure 5.13: Shear stress versus displacement for Type 7 (hollow facing unit with steel pin and NCA)

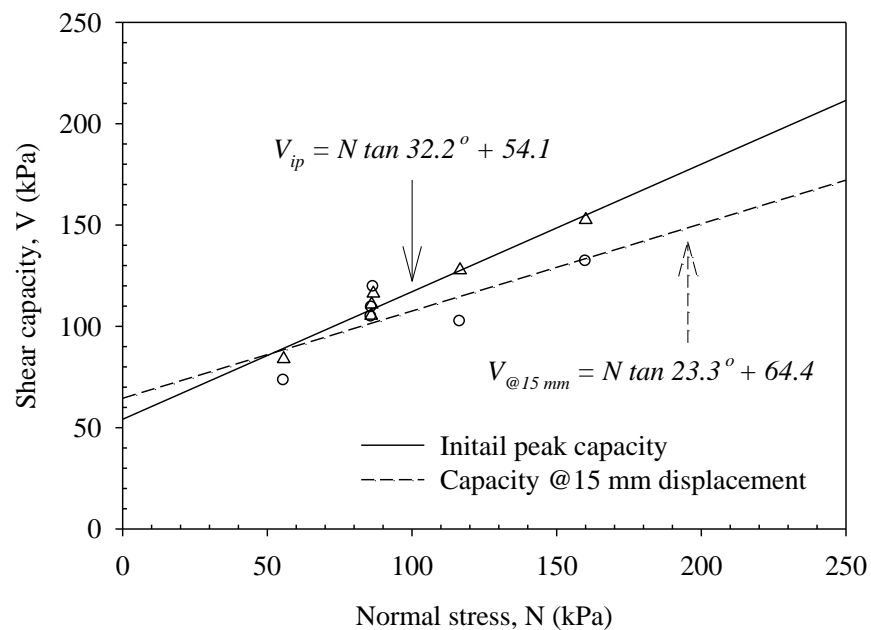


Figure 5.14: Interface shear capacity versus normal stress for Type 7 (hollow facing unit with steel pin and NCA)

5.3.6 Type 8 (Concrete-to-concrete interface with plastic pin and granular infill)

Type 8 identifies the interaction behavior between I-Block system infilled with NCA and plastic pins (mechanical connectors) used as shear transferring device at block interface. Figure 5.15 illustrates the curves of shear strength against displacement under different normal loads imposed and recorded during interface shear testing. From the Figure 5.15, it is seen that shear stress increases gradually without any peak as found in Figure 5.13 and finally reached the maximum value after a significant amount of displacement (roughly about 20 mm). To compare the shear capacities of this type of configuration under peak and serviceability conditions, the shear stress data corresponding normal stresses are presented in Figure 5.16. From the shear capacity equations displayed in Figure 5.16, it can be seen that both of the serviceability performance parameters are lower than peak performance parameters.

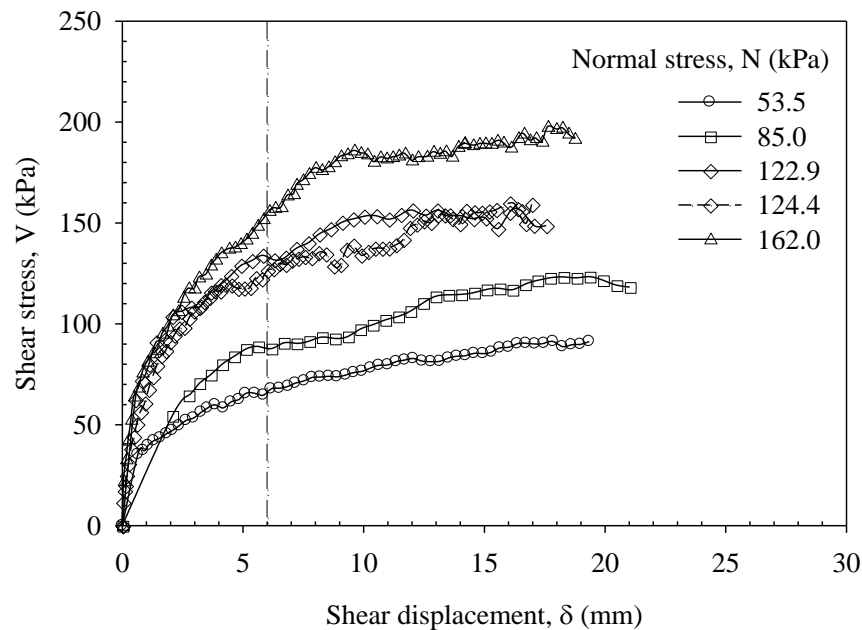


Figure 5.15: Shear stress versus displacement for Type 8 (hollow facing unit with plastic pin and NCA)

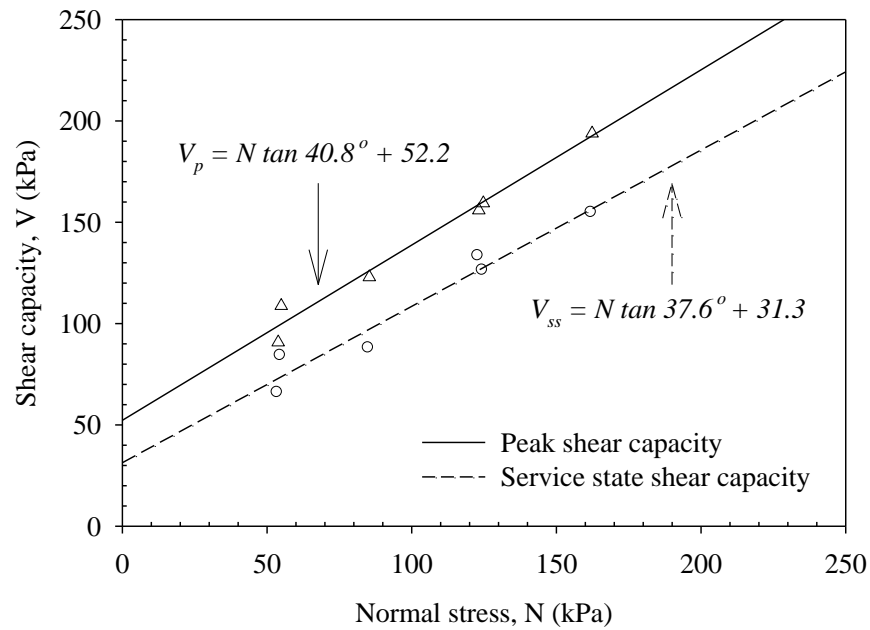


Figure 5.16: Interface shear capacity versus normal stress for Type 8 (hollow facing unit with plastic pin and NCA)

5.4 Group 3: Effect of flexibility of geosynthetic inclusion on interface shear capacity

5.4.1 Overview

This group of tests configures nine series of interface shear tests consisting of Types 9 to 17. The tests were configured depending on the geosynthetic inclusions and granular in-fills. The primary objective of this group was to determine the performance parameters of the new block system with interlocking materials (plastic pins and all types of granular in-fills) and geosynthetic inclusions. This group represents the potential field conditions of reinforced I-Block walls with proposed interlocking materials. Types 9 to 11 present the data of interface shear testing under PET geogrid (#1) inclusion with different types of in-fills. Frictional performance of infilled I-Blocks with HDPE geogrid (#2) inclusion is included in Types 12 to 14. Last three Types (15 to 17) present the interface shear behavior with PET geotextile inclusion.

5.4.2 Type 9 (Concrete-PET geogrid-concrete interface with plastic pin and NCA infill)

Type 9 was configured to investigate the frictional performance of infilled segmental concrete units with plastic pins and a single layer of polyester (PET) geogrid inclusion which is flexible in nature. Natural coarse aggregate (NCA) was used in Type 9 to compare the frictional behavior of PET geogrid with other types of granular in-fills used as alternative of NCA, e.g. recycled coarse aggregates (RCA). The data of this series of tests are presented in Figure 5.17 as form of shear stress-displacement relationships at various normal stresses. The plots of the Figure 5.17 show increasing of shear resistance gradually without any peak with shear displacement up to a significant limit of about 20 mm. The peak and service state shear capacities of this configuration is outlined in Figure 5.18. The performance parameters presented in Figure 5.18 define peak shear capacity envelope which is quite higher than serviceability condition. It can be seen that serviceability envelope moves downward relative to peak envelope with increasing normal stress. This may be happened due to the caution effect of flexible geogrid reinforcement. The results of Type 9 are chosen as referenced data for the following two Types (10 and 11).

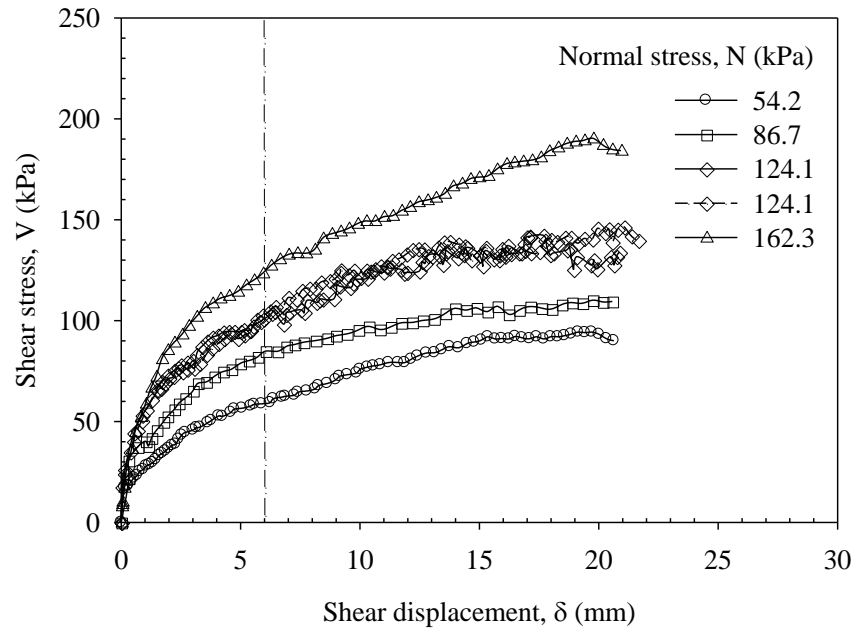


Figure 5.17: Shear stress versus displacement for Type 9 (hollow facing unit with plastic pin, NCA and PET geogrid inclusion)

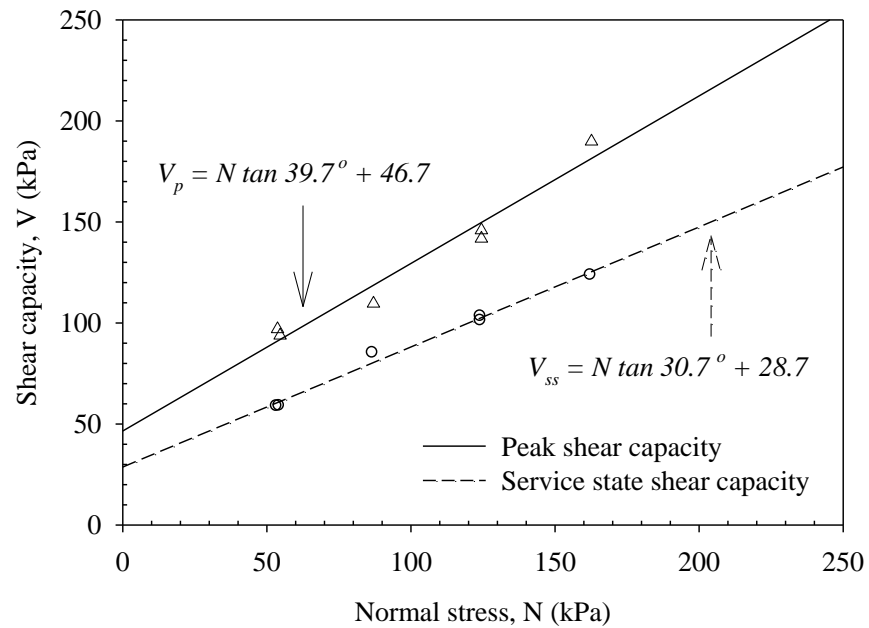


Figure 5.18: Interface shear capacity versus normal stress for Type 9 (hollow facing unit with plastic pin, NCA and PET geogrid inclusion)

5.4.3 Type 10 (Concrete-PET geogrid-concrete interface with plastic pin and RCA 1 infill)

This test series evaluate the frictional behavior of RCA 1 infilled modular block system with plastic pins and a single layer of PET geogrid (flexible) inclusion at interface. The test results of this configuration are plotted by means of shear stress-displacement and shear stress-normal stress relationships. Figure 5.19 shows shear stress-displacement curves including repeated tests. The curves show gradual variation of shear stress in magnitude and distribution against lateral displacement. From the Figure 5.19, it is seen that the shear stress-displacement plot for a normal stress of about 86 kPa moved towards the plot of lower imposed normal load (about 52 kPa) after quite enough relative displacement. This may be happened due to compaction of RCA 1, which contains more void contents. Comparison of interface shear capacities of the Type 10 configuration under peak and service state criteria is outlined in Figure 5.20. It is seen that the Figure 5.20 shows the similar of shear capacity envelopes as described earlier in Type 9.

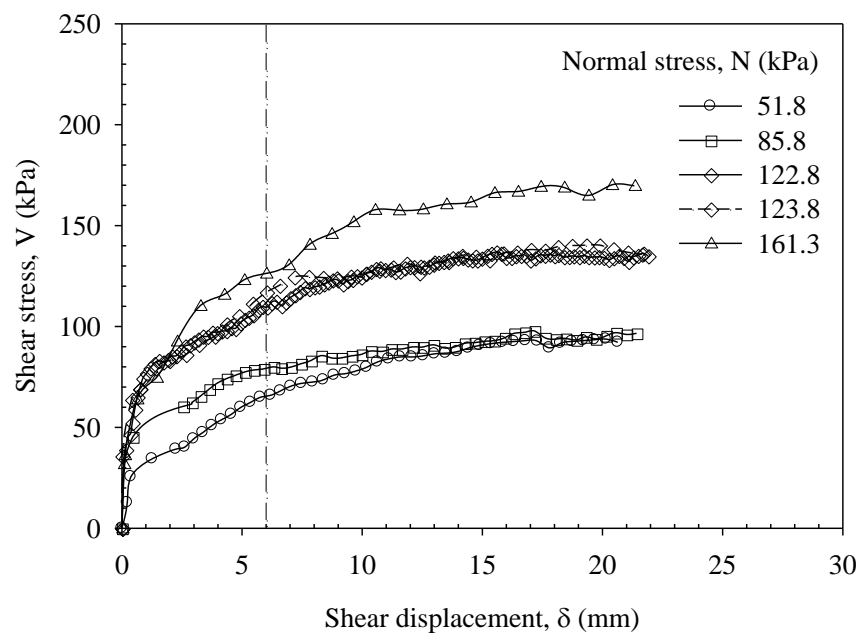


Figure 5.19: Shear stress versus displacement for Type 10 (hollow facing unit with plastic pin, RCA 1 and PET geogrid inclusion)

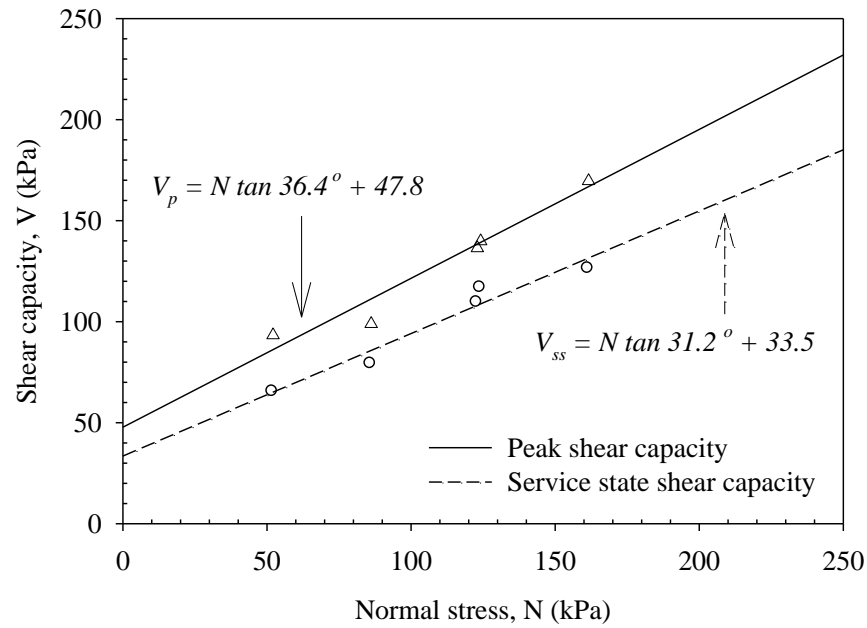


Figure 5.20: Interface shear capacity versus normal stress for Type 10 (hollow facing unit with plastic pin, RCA 1 and PET geogrid inclusion)

5.4.4 Type 11 (Concrete-PET geogrid-concrete interface with plastic pin and RCA 2 infill)

In comparison with Types 9 and 10, RCA 2 was used as a granular infill in this configuration to investigate its performance with a PET geogrid inclusion. Figure 5.21 demonstrates the shear stress-displacement curves at different surcharge pressures. The plots of Figure 5.21 illustrate the same typical shape as seen in Figure 5.19 regardless the curve at a normal stress of about 86 kPa. For determining design envelopes for this type of wall system, a comparison of peak and serviceability shear envelopes are shown in Figure 5.22. It can be seen that the capacity envelopes follow the same typical pattern as found in Types 9 and 10.

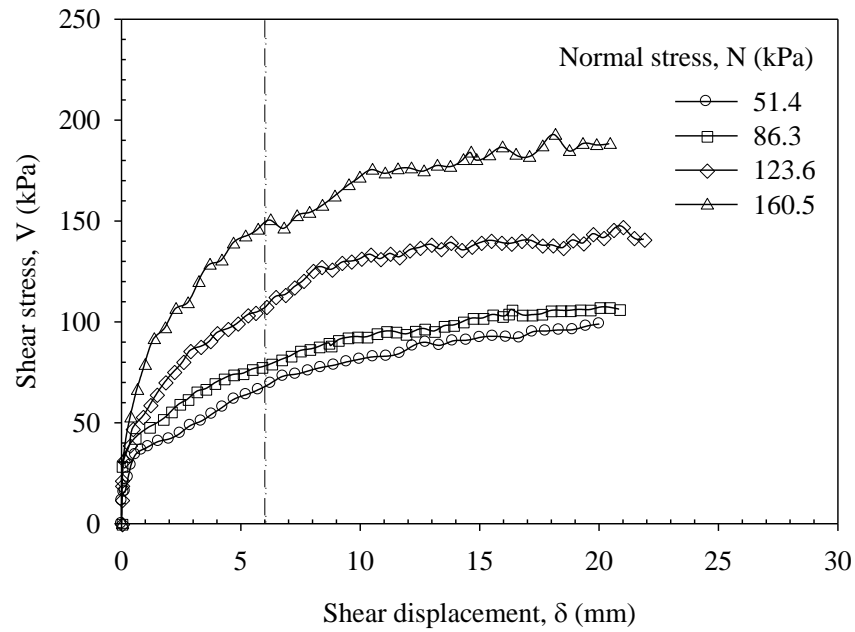


Figure 5.21: Shear stress versus displacement for Type 11 (hollow unit with plastic pin, RCA 2 and PET geogrid inclusion)

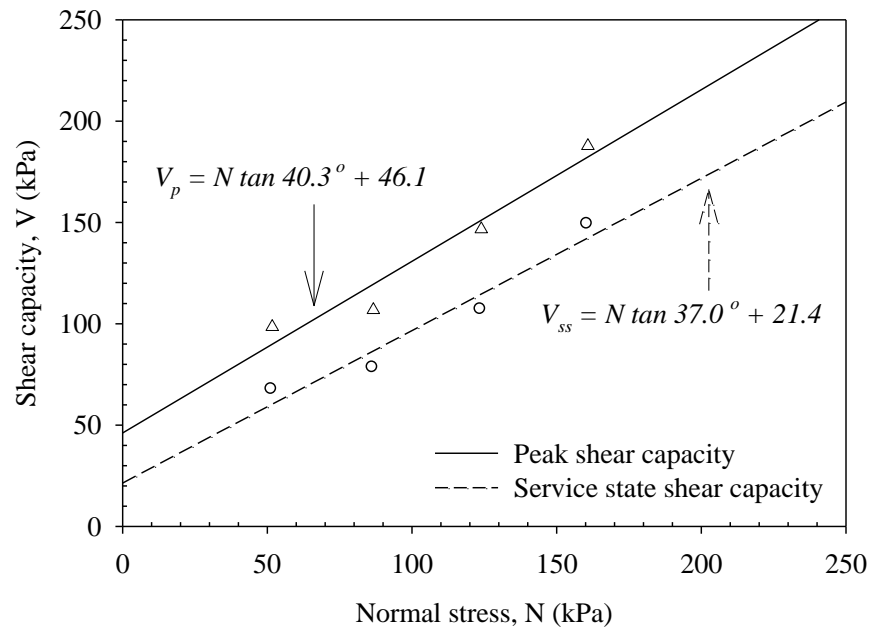


Figure 5.22: Interface shear capacity versus normal stress for Type 11 (hollow facing unit with plastic pin, RCA 2 and PET geogrid inclusion)

5.4.5 Type 12 (Concrete-HDPE geogrid-concrete interface with plastic pin and NCA infill)

This test series was configured to investigate interface shear behavior of infilled segmental concrete units with fresh (natural aggregate), and with plastic pins and a single layer of a HDPE geogrid (stiff) inclusion. Shear stress-displacement curves of Type 12 under different surcharge loading conditions are drawn in Figure 5.23. It is, therefore, seen that shear stress increases gradually without any notable peak against lateral displacement and finally reached to the maximum value after a significant amount of displacement. To compare the interface shear capacity under peak and service state criteria, a shear stress versus normal stress graph is given in Figure 5.24. It can be seen that although service state shear capacity less than peak shear capacity but the serviceability angle of friction is higher than peak angle of friction. Therefore, serviceability envelope moves upwards relative to the peak envelope with increase in normal stress. This may be happened due to the presence of thick bond (4.1 mm) of HDPE geogrid at the interface. The data of Type 12 are considered as referenced data for the Type 13 and 14 configurations.

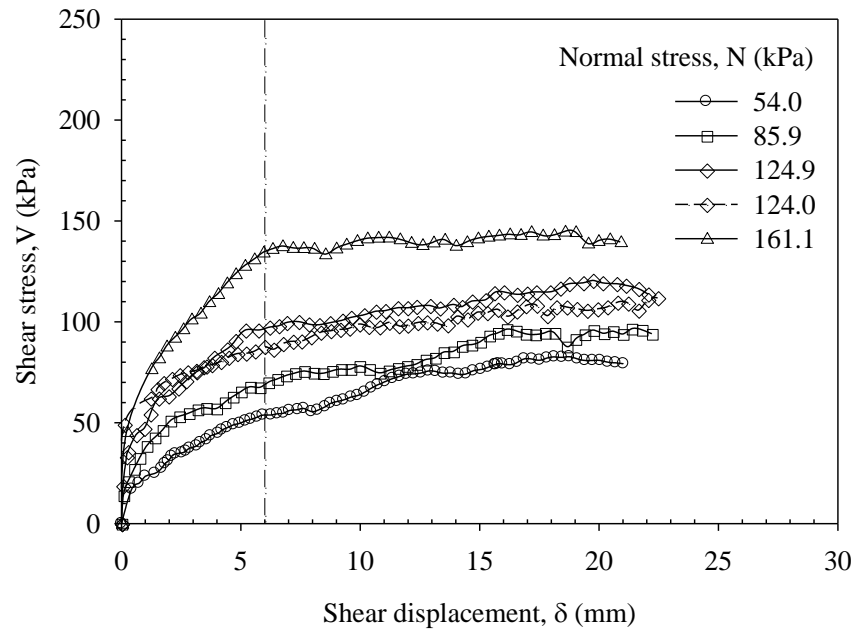


Figure 5.23: Shear stress versus displacement for Type 12 (hollow facing unit with plastic pin, NCA and HDPE geogrid inclusion)

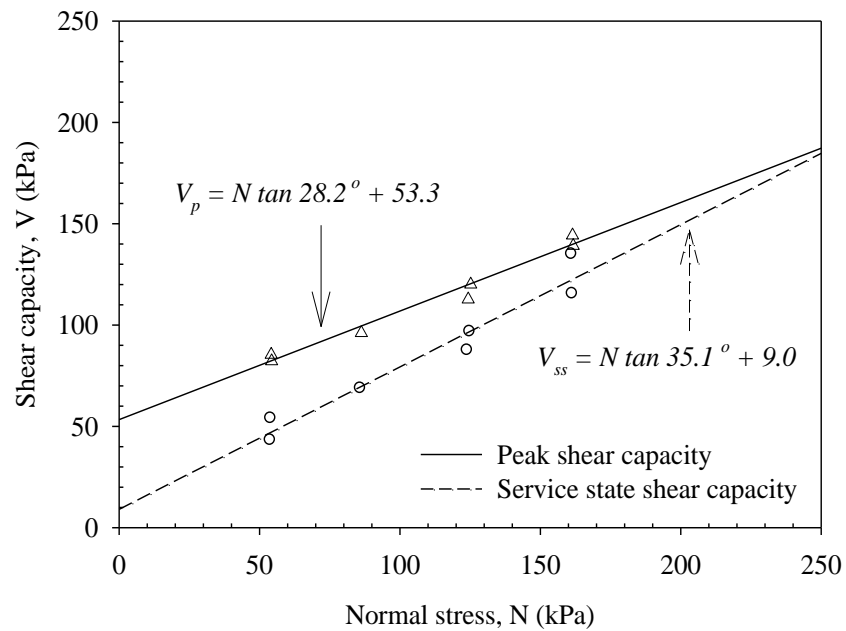


Figure 5.24: Interface shear capacity versus normal stress for Type 12 (hollow facing unit with plastic pin, NCA and HDPE geogrid inclusion)

5.4.6 Type 13 (Concrete-HDPE geogrid-concrete interface with plastic pin and RCA 1 infill)

A series of tests were included in configuration of Type 13 to study performance parameters of hollow segmental concrete units infilled with recycled aggregate (RCA 1) and along with plastic pins and a stiff extruded geogrid inclusion. Figure 5.25 displays the plots relating shear stress and displacement of the tested block systems for various normal stresses. The curves of Figure 5.25 follow the similar behavior as described in Figure 5.23. By comparing Figure 5.25 with Figure 5.19, it can be seen that rising of shear stress with the increment of normal stress is low and therefore, the curves are very close to each other and congested. It may be resulted from the presence of stiff geogrid at interface, which mobilize of top block easily. Figure 5.26 illustrates the interface shear capacity under ultimate (peak) and service state criteria. The plots show the similar behavior as reported in Figure 5.24.

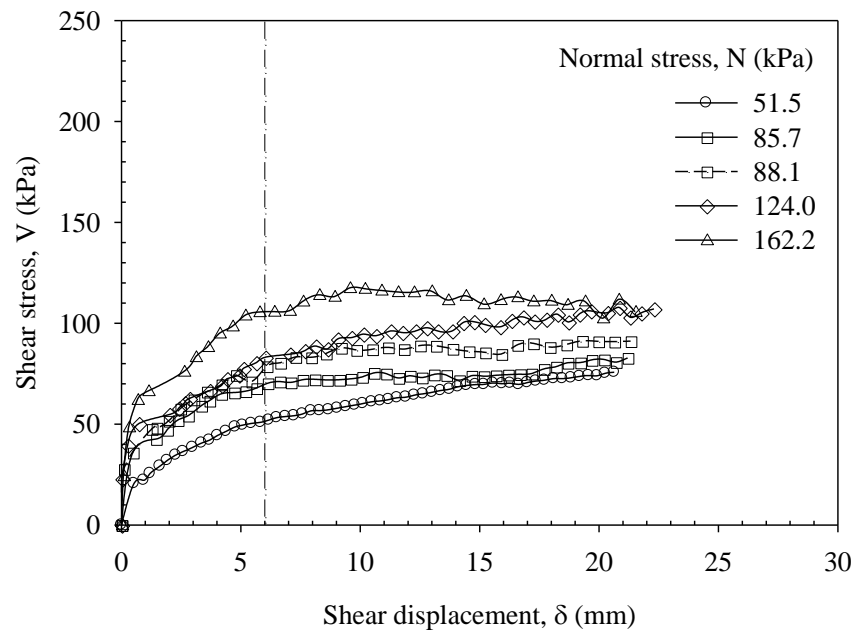


Figure 5.25: Shear stress versus displacement for Type 13 (hollow facing unit with plastic pin, NCA and HDPE geogrid inclusion)

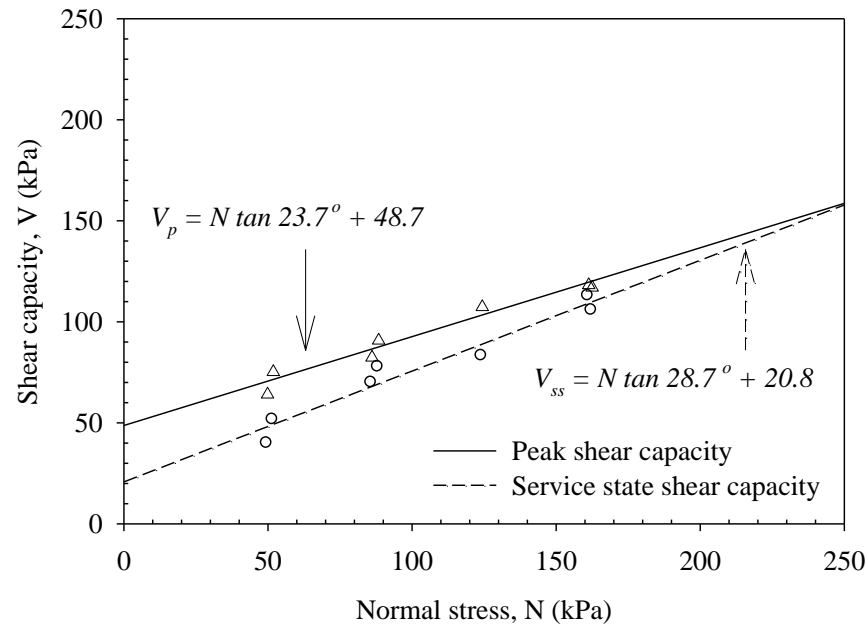


Figure 5.26: Interface shear capacity versus normal stress for Type 13 (hollow facing unit with plastic pin, RCA 1 and HDPE geogrid inclusion)

5.4.7 Type 14 (Concrete-HDPE geogrid-concrete interface with plastic pin and RCA 2 infill)

Type 14 was configured to study interface shear strength of hollow modular units infilled with recycled aggregate (RCA 2), and along with plastic pins and a single layer inclusion of stiff extruded geogrid (uniaxial). Figure 5.27 illustrates the curves of shear stress against lateral displacement under a range of normal loading levels. The curves show the similar pattern as mentioned in Figure 5.23. A comparison between peak and serviceability shear capacities for Type 14 is shown in Figure 5.28. The service state shear capacity envelope follow the similar pattern with respect to peak shear capacity envelop as found in Figures 5.24 and 5.26. The envelope lines are very close to parallel although there is a slight difference in between the friction angles.

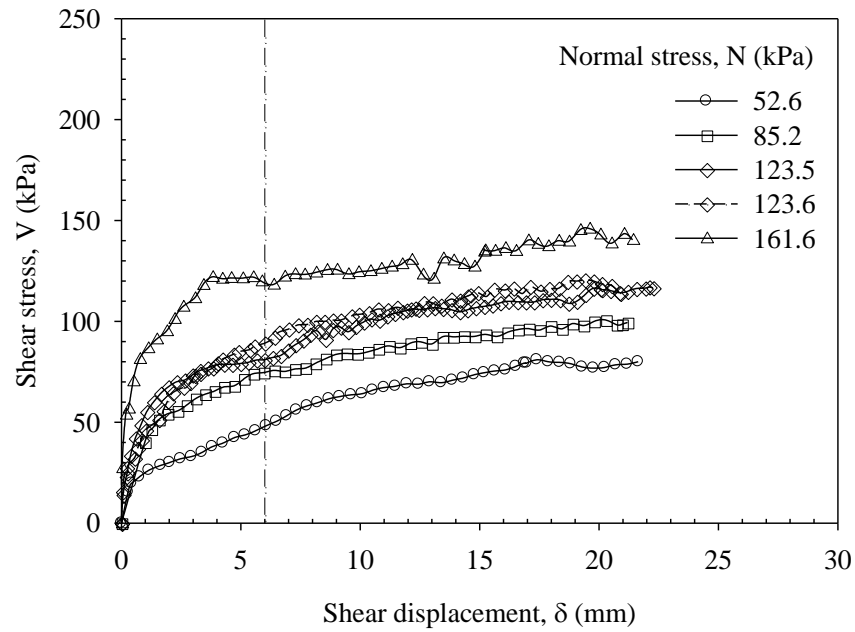


Figure 5.27: Shear stress versus displacement for Type 15 (hollow facing unit with plastic pin, RCA 2 and HDPE geogrid inclusion)

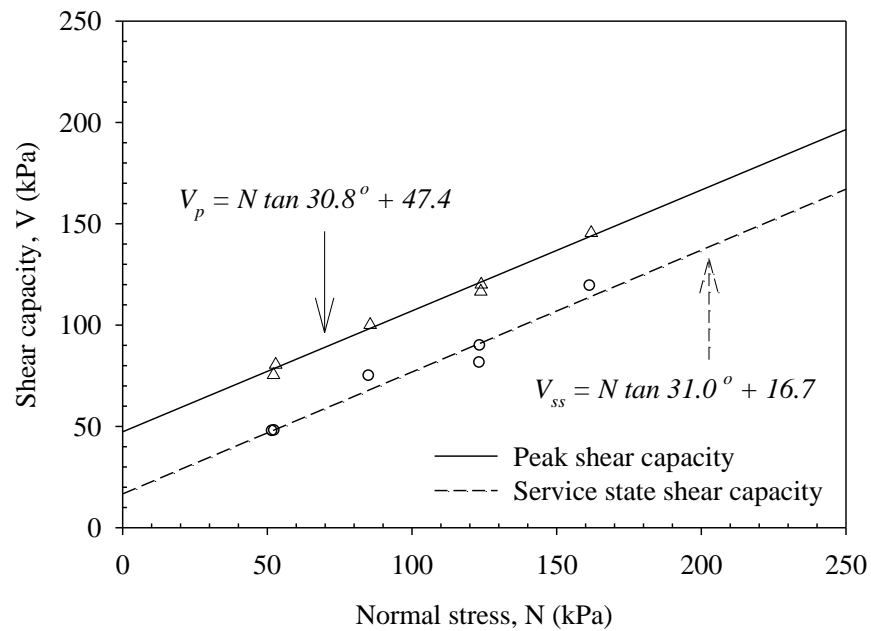


Figure 5.28: Interface shear capacity versus normal stress for Type 14 (hollow facing unit with plastic pin, RCA 2 and HDPE geogrid inclusion)

5.4.8 Type 15 (Concrete-PET geotextile-concrete interface with plastic pin and NCA infill)

Type 15 includes a series of test performed to investigate the interface shear capacity of infilled hollow segmental retaining wall units, and with plastic pins and flexible non-woven geotextile inclusion. As a granular infill, fresh (natural) aggregate was selected in this test series. The test results of this series were plotted in the forms of shear stress-displacement and shear stress-normal stress relationships. Figure 5.29 compares the magnitude and distribution of shear stress-displacement curves for different normal stresses. The curves show increasing of shear resistance against lateral displacement up to a certain amount of displacement and then decrease very mildly for certain amount of normal stresses. The data of shear stresses against different normal loads are plotted in Figure 5.30 to identify shear capacity envelopes for peak and serviceability criteria. It is seen that the service state performance parameters are lower than peak criterion and the envelopes are almost parallel. The results of Type 15 are used as referenced data for Types 16 and 17, where recycled aggregates (RCA 1 and 2) were used as granular infills.

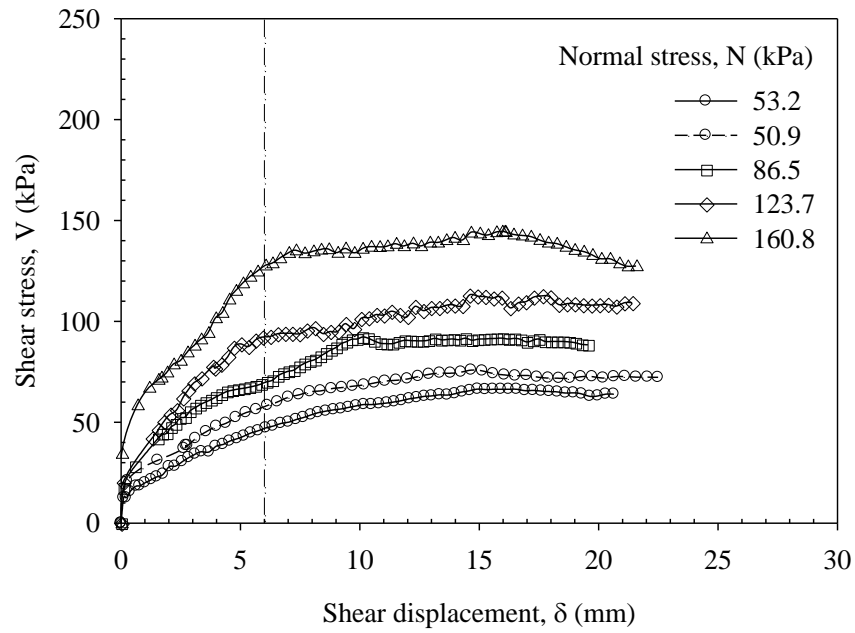


Figure 5.29: Shear stress versus displacement for Type 15 (hollow facing unit with plastic pin, NCA and PET geotextile inclusion)

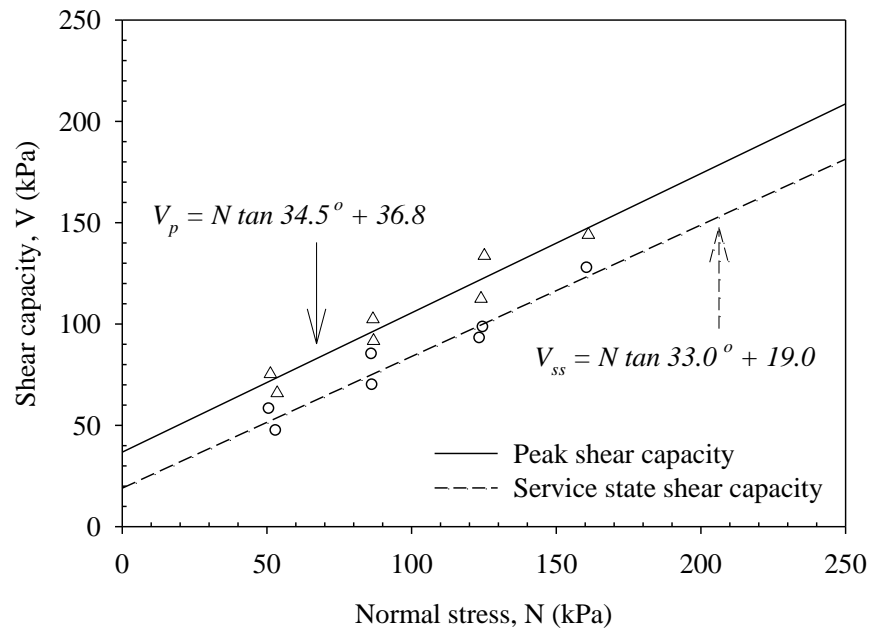


Figure 5.30: Interface shear capacity versus normal stress for Type 15 (hollow facing unit with plastic pin, NCA and PET geotextile inclusion)

5.4.9 Type 16 (Concrete-PET geotextile-concrete interface with plastic pin and RCA 1 infill)

Type 16 focuses on interface shear testing of infilled concrete units with plastic pins and a flexible geotextile inclusion. In this test series, recycled aggregate (RCA 1) was used to compare its frictional performance against NCA with a flexible geotextile inclusion. The magnitude and distribution of shear stress with displacement for various surcharge pressures including repeated tests is shown in Figure 5.31. The curves show gradually increasing of shear stress with increasing lateral displacement. Interface shear capacities under peak and deformation (serviceability) criteria for this test series is reported in Figure 5.32. The envelope lines show that they are stepping aside from each other with increasing normal stress.

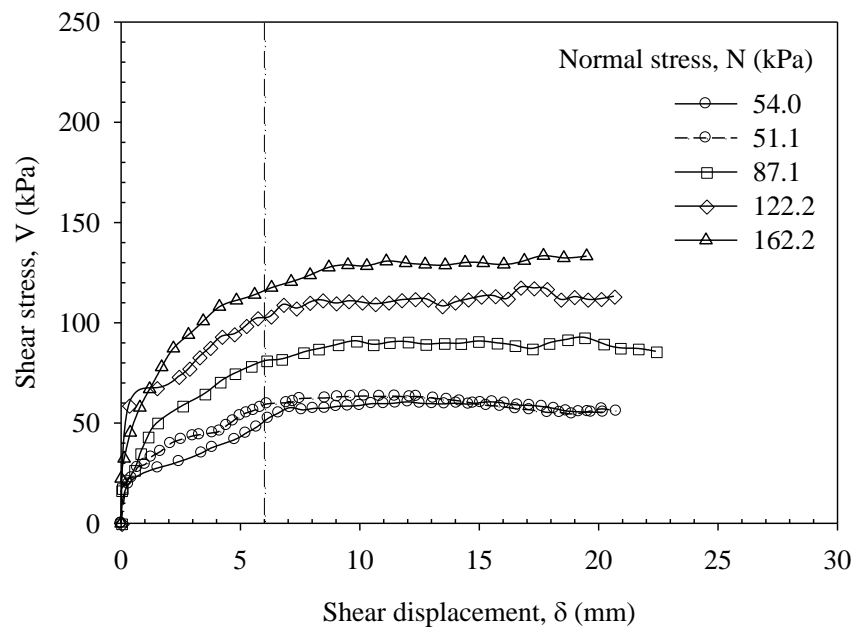


Figure 5.31: Shear stress versus displacement for Type 16 (hollow facing unit with plastic pin, RCA 1 and PET geotextile inclusion)

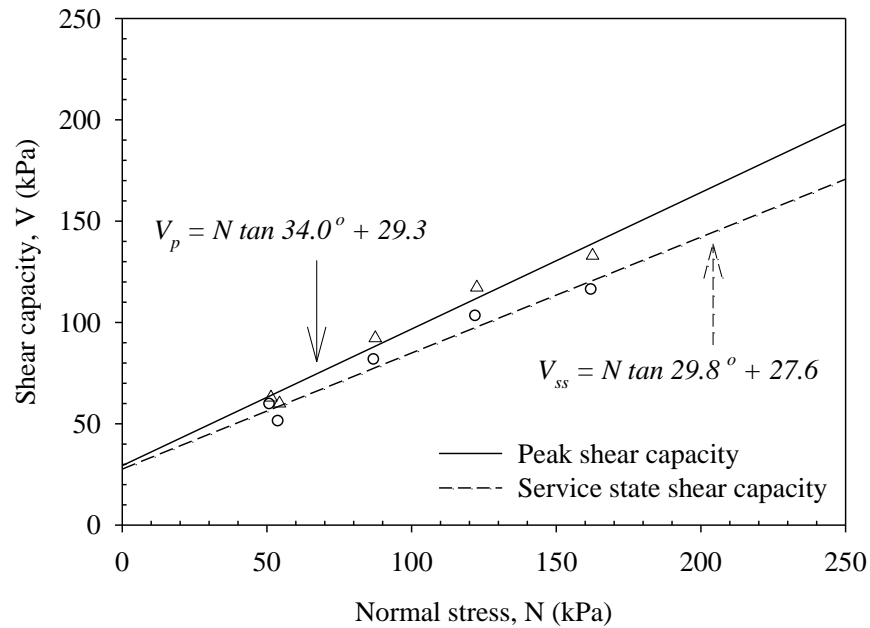


Figure 5.32: Interface shear capacity versus normal stress for Type 16 (hollow facing unit with plastic pin, RCA 1 and PET geotextile inclusion)

5.4.10 Type 17 (Concrete-PET geotextile-concrete interface with plastic pin and RCA 2 infill)

A series of tests for Type 17 were performed to investigate the interface shear strength of I-Block system infilled with recycled aggregate (RCA 2) and with plastic pins and a layer of inclusion of a polyester (PET) geotextile. Figure 5.33 demonstrate the curves relating shear stress and displacement under different normal stress conditions. Figure 5.33 also shows repeated shear stress-displacement curves under a normal stress of about 53 kPa. For brevity and better presentation, only selected curves of this test series are presented here. The interface shear capacity envelopes of this test series are plotted in Figure 5.34. Shear capacities envelopes show a comparative evaluation of shear strength under peak and deformation criteria. For shear capacity envelopes, all repeated test data are considered to represent the results of repeated tests those which peak shear stresses are less than $\pm 10\%$ from the mean of the tests.

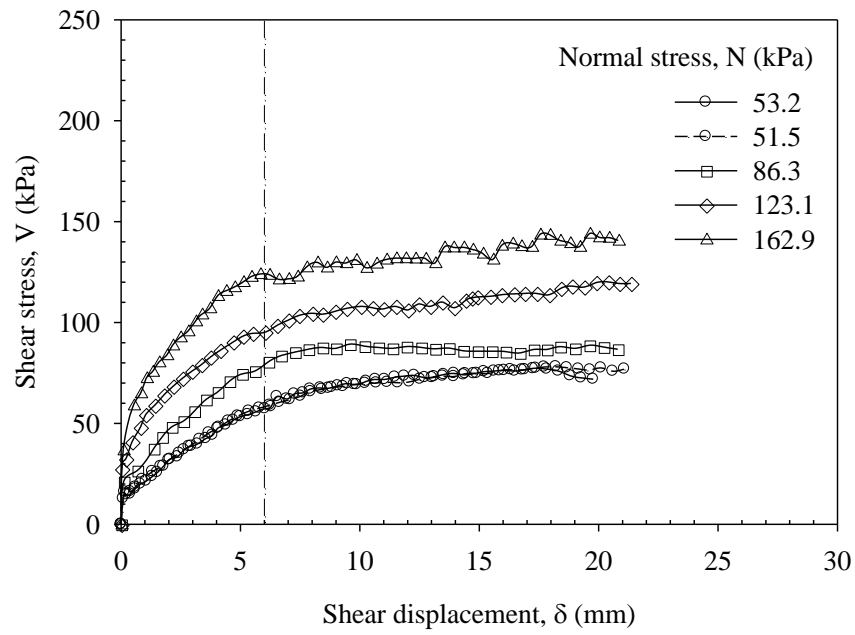


Figure 5.33: Shear stress versus displacement for Type 17 (hollow facing unit with plastic pin, RCA 2 and PET geotextile inclusion)

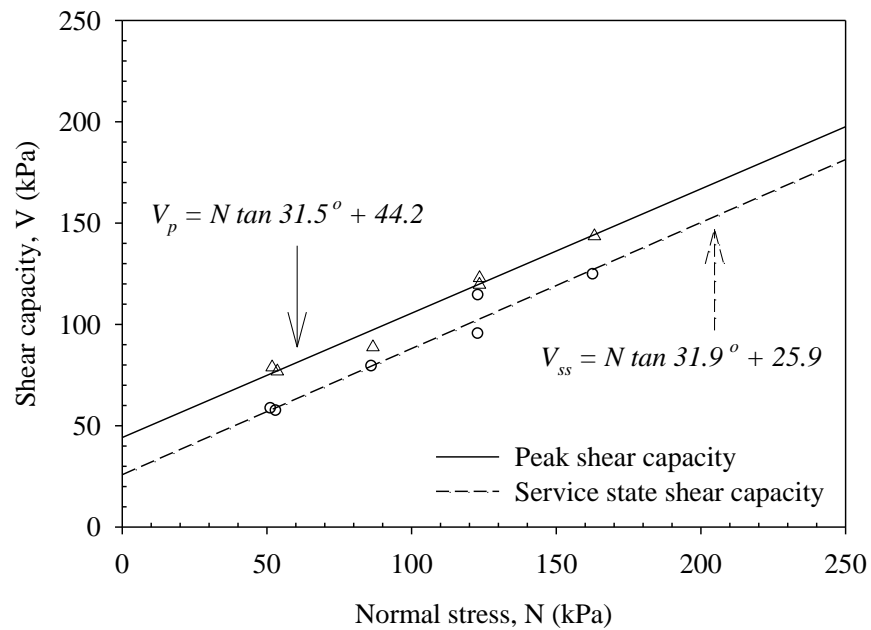


Figure 5.34: Interface shear capacity versus normal stress for Type 17 (hollow facing unit with plastic pin, RCA 2 and PET geotextile inclusion)

CHAPTER 6 DISCUSSIONS

6.1 General

The results of different configurations of tests (Types 1 to 17) have been presented as summarized form in Chapter 5. Chapter 6 provides a comparison and discussion of the selected data collected from the configurations to find out the effects of mechanical connectors, recycled aggregates and geosynthetic inclusion on interface shear capacity.

6.2 Effect of stiffness (rigidity) of shear pin on interface shear capacity of facing units

The results of Types 2 and 3 configurations were compared with Type 1 of zero stiffness (no shear pins) to evaluate the effect of pin's rigidity on the shear strength of the tested blocks. The comparison of the tests data is done by plotting shear stress-displacement and shear stress-normal stress relationships under peak (ultimate) criterion.

The curves of Figures 6.1 to 6.3 illustrate the frictional behavior of the hollow block system for different surcharge (normal) pressures and different types of shear pins of different rigidities. The variation in normal stress increments among the test series was due to the manual controlling of normal pressure by using a pressure adjustment knob.

For the purely friction condition (without shear connectors), the curves illustrate the rapid increase of shear stresses at the early stage of load application. It may be happened due to frictional resistance of plain concrete surfaces. After reaching the maximum shear resistance, it heads towards almost a constant value with the mobilization of block.

Although, the curves show abrupt rise and fall of shear stresses with displacement for high normal stresses that may be resulted from frictional interlocking of irregular contact areas at block's interface. At the time laboratory testing, it was also observed that sudden fall of shear stresses happens due to insignificant spalling of flanges (front and rear) of top block at the interface. The spalling patters of top block are shown in Figure A.2.

The increasing patterns of shear stress at the beginning for test with plastic pins show almost similar patterns as described for purely frictional conditions but shear strength heading towards a higher maximum value with the lateral displacement and then decrease gradually with the mobilization of block. Although, the initial shear resistance is controlled by concrete-to-concrete surface friction, the presence of flexible shear pins provides additional shear resistance to the block interface. From the curves for plastic pins, it is seen that shear resistance drops gradually after a significant amount of displacement and heading towards purely frictional shear resistance. It occurs due to the pure shear failure of flexible connectors after certain amount of displacement. Figure A.3 shows the failure patterns of flexible plastic pins.

For the tests with steel pins, the curve shows a typical saw-tooth pattern of shear stress after a small amount of displacement. Due to the presence of rigid pins (steel), shear stress increases sharply to a lower maximum value and then drops insignificantly. It was observed that the small drop of shear stress corresponded to the initiation of cracks at the running joints and/or insignificant spalling at flanges and joints. Due to progressive failure patterns of blocks, shear stress reaches the higher maximum value and then drops significantly (Figure 6.2).

The cracks at the running joints of blocks occurred due to stress concentration generated by steel (rigid) pins and propagate with displacement. In some cases, it was observed that both joints do not fail together due the block setup and block geometry. As a result, after failure of one joint shear resistance increase again and dropped permanently after complete failure of both joints (Figures 6.1 and 6.3). Due to the high stiffness, steel pin does not fail in shear but just bends slightly at high shear force. Bending of steel pins and failure of concrete blocks at joint are shown in Figures A.4 and A.5.

The plots presented in Figure 6.4 illustrate the peak (ultimate) shear capacity envelopes for different types of shear connectors with different flexibilities. It is clear from the Figure 6.4 that the shear capacities of blocks with shear connectors are higher than those without shear connectors (purely frictional interface). Bathurst and Simac (1997) and Bathurst et al. (2008) reported the similar effects of mechanical interlocks or connectors on interface shear capacity for different type of block geometries. The initial peak capacity of block with steel pins is relatively higher at low surcharge pressures but the capacity significantly reduced at high normal stress than those with plastic pins. This happened due to rigidity and strength of steel pins that caused the concrete to break at small displacement (<6 mm) and reduced the area of contact significantly at high normal stress although rigid pins provided a higher apparent cohesion (normal stress-independent shear strength) than flexible plastic pins.

Shear pins are one type of mechanical connectors used to align the blocks and to provide additional interlocking to the wall system as well. If they are too rigid and strong it can damage the block at relatively small displacement and consequently reduce the interface shear capacity.

It can be said that shear pins especially flexible pins deliver more effective shear connection than purely frictional interfaces and even rigid pins (steel).

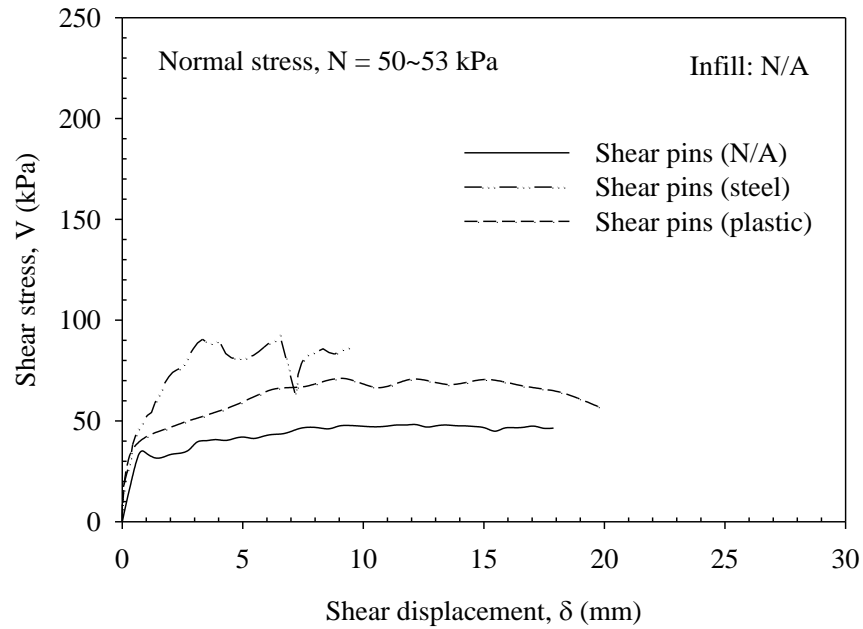


Figure 6.1: Shear stress versus displacement (hollow facing unit with different types of shear pins)

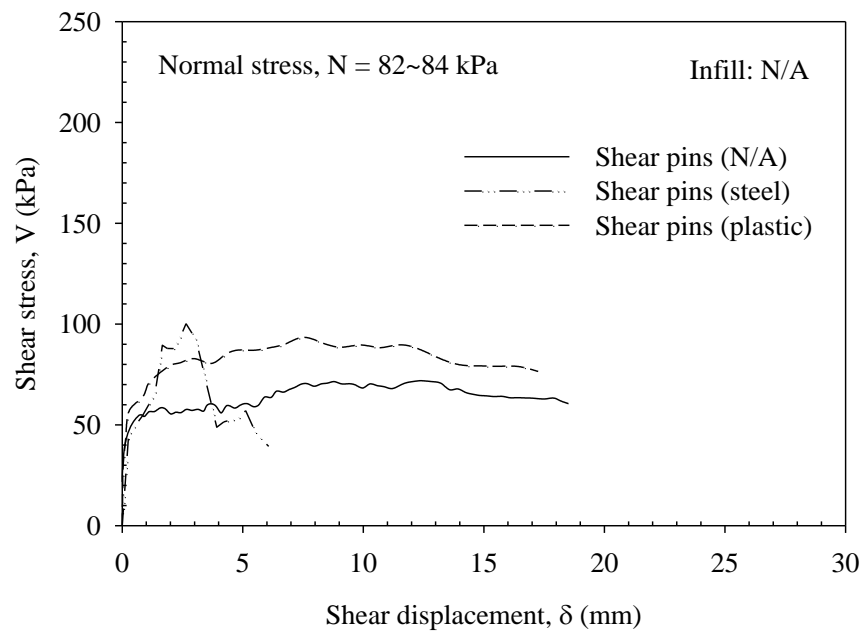


Figure 6.2: Shear stress versus displacement (hollow facing unit with different types of shear pins)

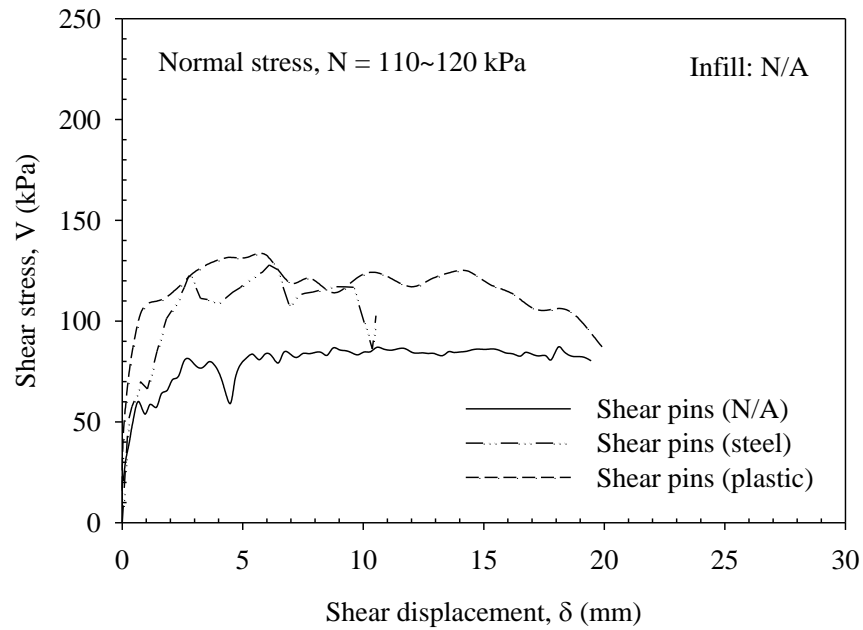


Figure 6.3: Shear stress versus displacement (hollow facing unit with different types of shear pins)

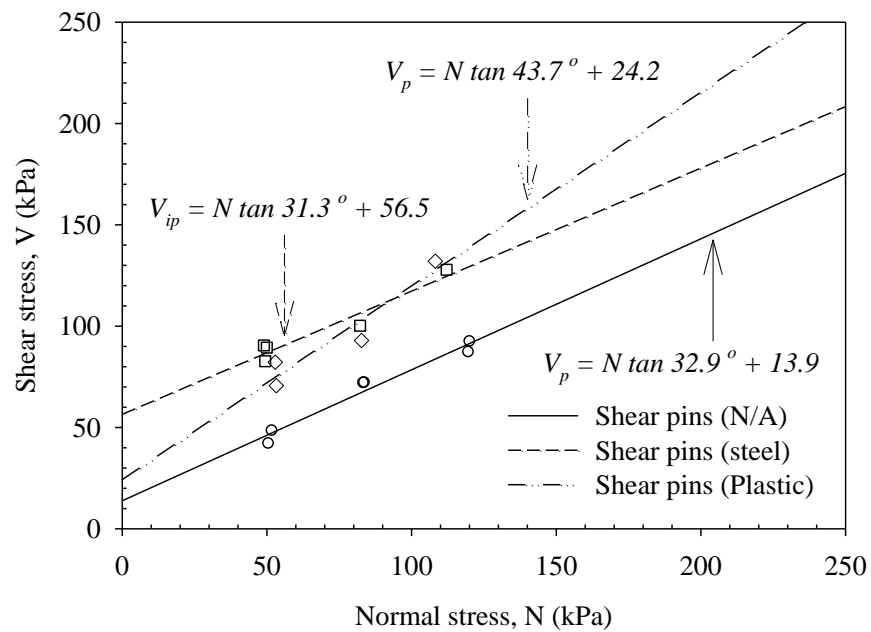


Figure 6.4: Interface shear capacity versus normal stress (hollow facing unit with different types of shear pins)

6.3 Frictional performance of hollow infilled concrete units interlocked with shear pins

To identify the effectiveness of shear pins, the results of Types 7 and 8 configurations were compared against Type 4. The tests data were outlined in the form of shear stress-displacement relationships to compare the influence of shear pins on shear strength of hollow infilled segmental concrete units. Shear capacity envelopes were also compared under peak criterion to compare the performance parameters for each case.

Figures 6.5 to 6.8 show the curves relating shear stress and lateral displacement of the tested segmental blocks for different normal stresses and different types of shear pins. For the tests without pins, the shear stress increases gradually without any peak and eventually reached the maximum value after a displacement of about 20 mm. More or less a similar behavior is shown for tests with flexible connectors but heading towards a higher maximum shear stress. However, for tests with rigid pins the curve shows a peak value at a displacement of about 4 mm after which the shear stress starts to decrease and some of them heading for a lower maximum stress than the one without connectors. It was observed that the peak value corresponded to the failure of the block at the joints. Once the joints have failed the interface capacity is only due the friction along the block contact surface and the infill aggregate. The reason why in some tests the interface resistance dropped so much is that there a significant reduction in contact surface as the block failed at the connections. In the case of the flexible connectors, the failure of the blocks did not occur and the plastic pins failed in shear instead (Figure A.6). The failure patterns of infilled blocks system with steel shear pins is show in Figure A.7.

Plots of the maximum shear stress against the applied normal stress for all types of shear pins are shown in Figure 6.9.

It can be seen that the shear capacity of infilled blocks with plastic pins is higher than those with steel pins. Because of its rigidity and strength the steel pins caused the concrete to break at certain displacement and reduced the area of contact. In the case of plastic pins, they did not break the concrete block and therefore no reduction of the area of contact. Mechanical connectors like shear pins are usually used to help align the blocks. If they are too rigid and strong it can damage the block at relatively small displacement and consequently reduce the interface capacity of the blocks. In practice the block should be allowed to move relative to one another as much as 6 mm. If the steel pins are used then there would a connection failure well before reaching this value of displacement as the tests indicated that failures happened at a displacement of only 4mm.

Although, plastic and steel pin were used as shear transferring device in this study to provide additional interlocking for the infilled block system but it is seen that plastic pin provides a better and effective interlocking to the block system in respect to shear strength and serviceability criterion.

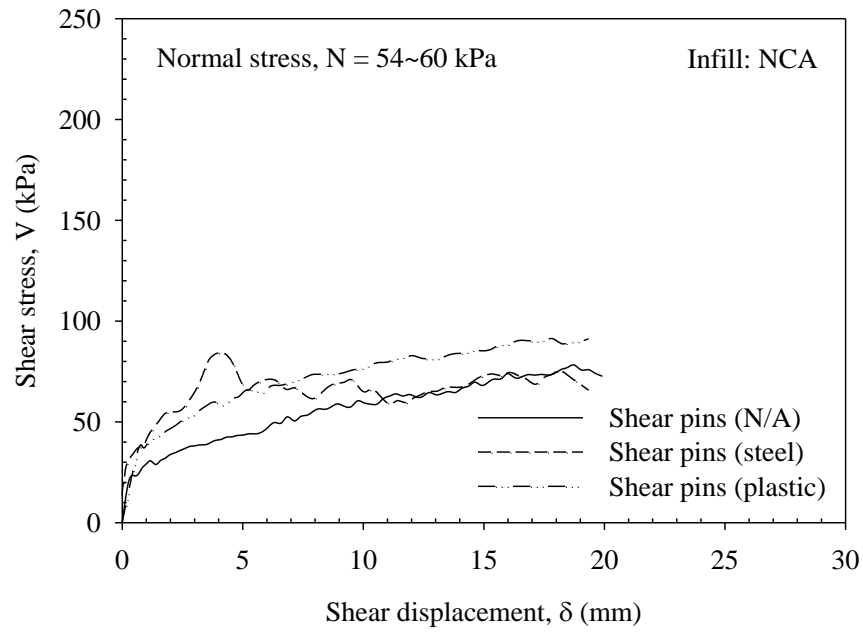


Figure 6.5: Shear stress versus displacement (hollow facing unit with different types of shear pins and NCA infill)

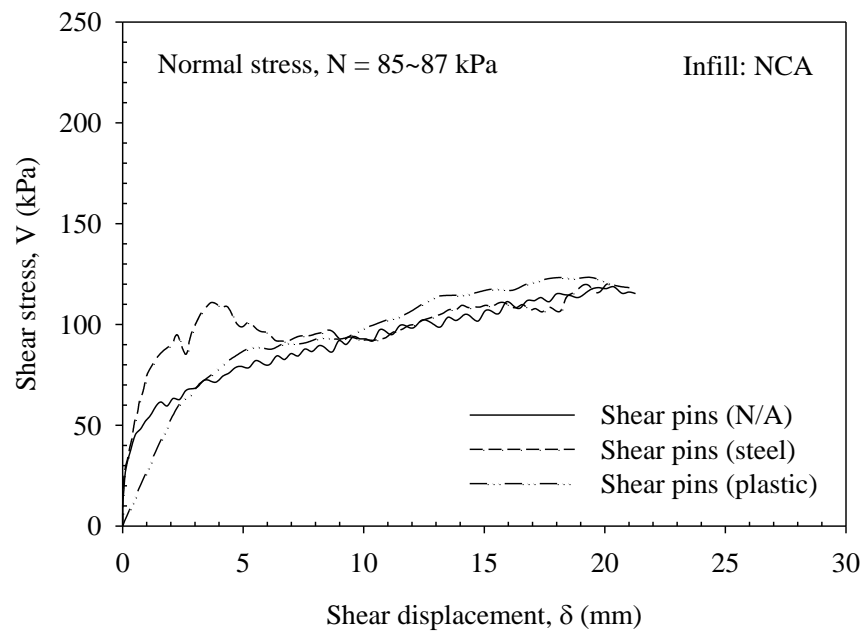


Figure 6.6: Shear stress versus displacement (hollow facing unit with different types of shear pins and NCA infill)

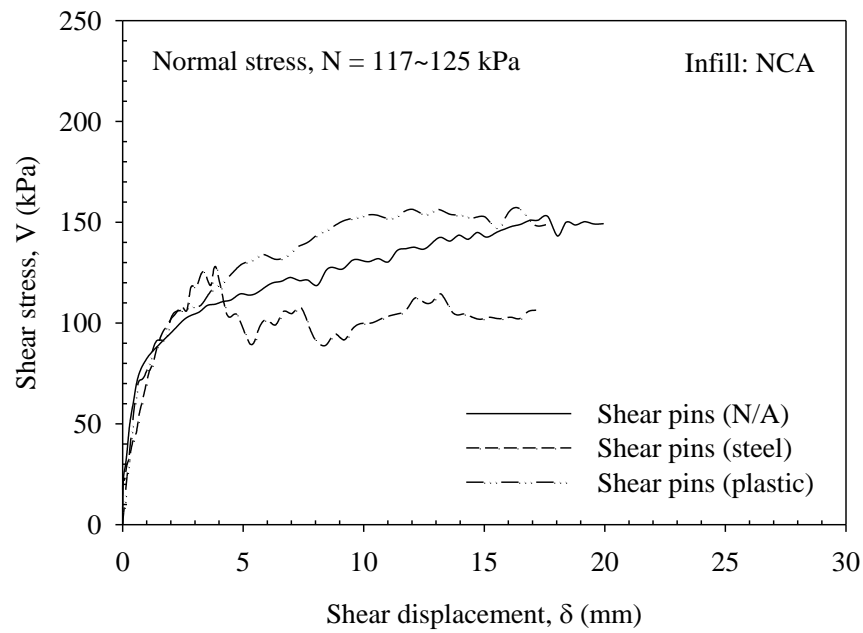


Figure 6.7: Shear stress versus displacement (hollow facing unit with different types of shear pins and NCA infill)

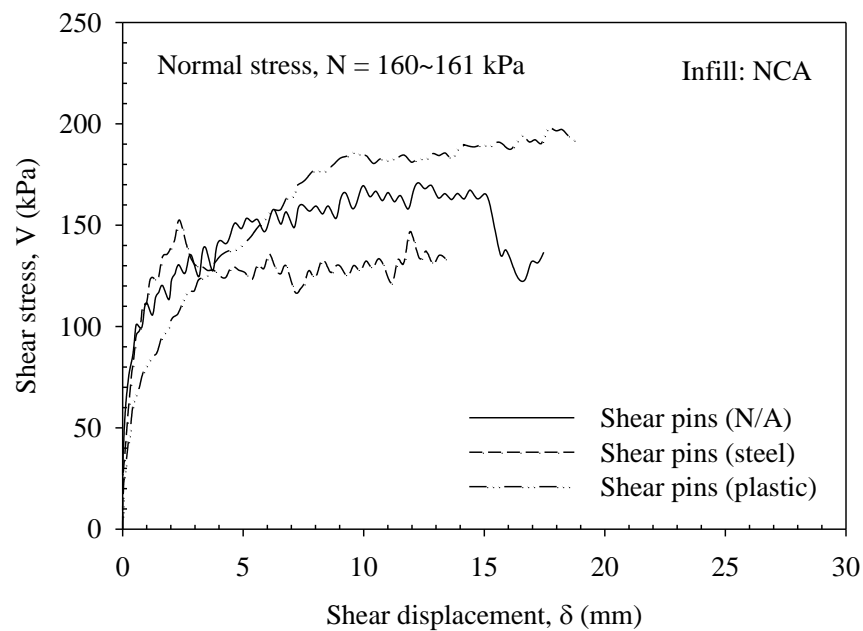


Figure 6.8: Shear stress versus displacement (hollow facing unit with different types of shear pins and NCA infill)

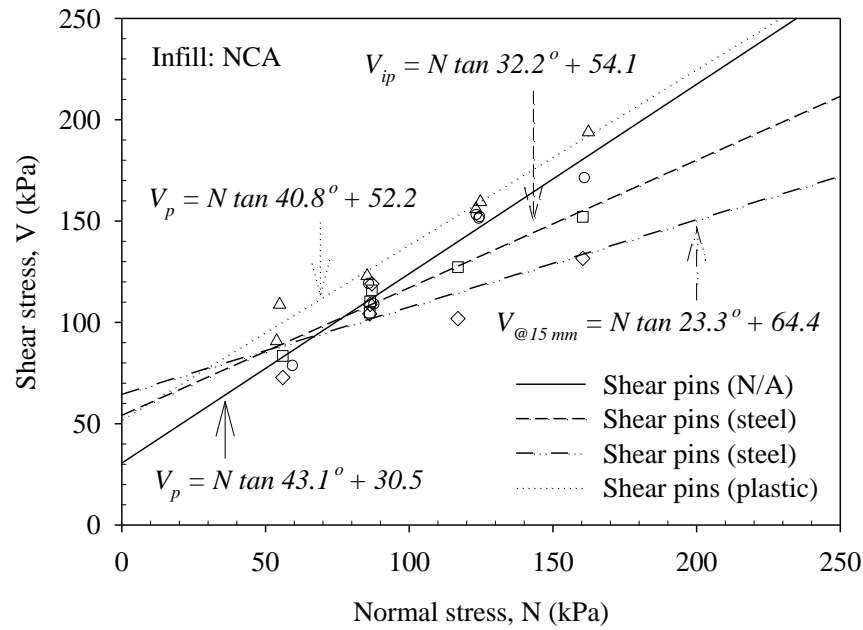


Figure 6.9: Interface shear capacity versus normal stress (hollow facing unit with different types of shear pins and NCA infill)

6.4 Effects of recycled aggregates used as granular in-fills on interface shear capacity of hollow modular block units

The results of Types 5 and 6 were compared with the referenced (control) configuration Type 4 in which natural coarse aggregate (NCA) was used as granular infill. Shear stress against displacement graphs were plotted to evaluate the effects of recycled coarse aggregates (RCA) on the interface frictional behavior of infilled blocks. Shear capacity envelopes were also plotted to compare the variation of interface shear capacity for hollow modular blocks infilled with different types of gravels used as infiller materials.

Figures 6.10 to 6.13 compare the frictional behavior of hollow concrete units infilled with different types of granular materials under different normal stresses.

For each type of combination, the shear stress increases gradually and reached the maximum (peak) value after a displacement of about 20 mm. It is also found that maximum shear stress for the hollow blocks infilled with recycled aggregates slightly lower than those with natural (fresh) aggregate. It may have happened due to the angularity and void content of the recycled aggregates used in this investigation. According to ASTM D5821, the granular materials used as in-fillers were 100% crushed and visually inspecting it was found that the fractured particles of recycled aggregates (RCA) were more angular and sharp at edges than NCA. The sharp edges of recycled aggregates consisting of cement-mortar mixture are relatively weaker than the edges of fresh aggregate. As a result, the weak sharp edges of recycled aggregates ruptured with the mobilization of block and ultimately reduced the shear strength because aggregates provide positive interlocking in the hollow block systems.

Figures 6.12 and 6.13 demonstrate the significant amount of rises and falls of shear stress through the displacement that makes the shear stress curves wavy than low normal stress (Figure 6.11). It may be resulted from stress concentration at the interface including concrete to concrete contact area and interlocking points of fractured particles (in-fills) because of high surcharge pressure. In these configurations of tests, the shear resistance of the infilled block system is governed by the presentence for granular in-fills that covers the 73 % of interface area. The sudden drops and falls of shear stresses may generally be related to the locking and unlocking of aggregate particles with each other during the testing and it continues with mobilization of blocks, and ultimate reached the maximum shear resistance after a displacement of about 20 mm. That is why the curves are wavy than purely frictional condition (Type 1) and especially for recycled aggregates those which have more sharp edges than NCA.

Figure 6.14 shows that the ultimate (peak) shear capacity envelopes for the hollow I-Block system infilled with different types of coarse aggregates. It can be said that peak shear capacities of the hollow block system infilled with recycled aggregates (RCA) almost equal those with natural aggregates. The performance parameters of the tested block system under peak criterion are summarized in Table 6.1. It is seen that NCA provides slightly higher angle of friction as well as apparent cohesion than RCA and this difference in shear strength can easily be ignored by considering sustainable development and waste minimization of concretes.

It can also be found that granular infill increases the interface shear capacity which is much higher than empty condition shown in Figure 6.4. Granular in-fills not only increase the angle of internal friction but also augment the apparent cohesion (normal-stress independent shear strength) of the system. This may be happened due to the interlocking mechanism of the crushed gravels, which enhances the positive interlock between the blocks and also increases the self-weight of hollow units. Guler and Astarci (2009) also reported that granular infill (gravel) increased the angle of friction for hollow segmental block system than other types of in-fills.

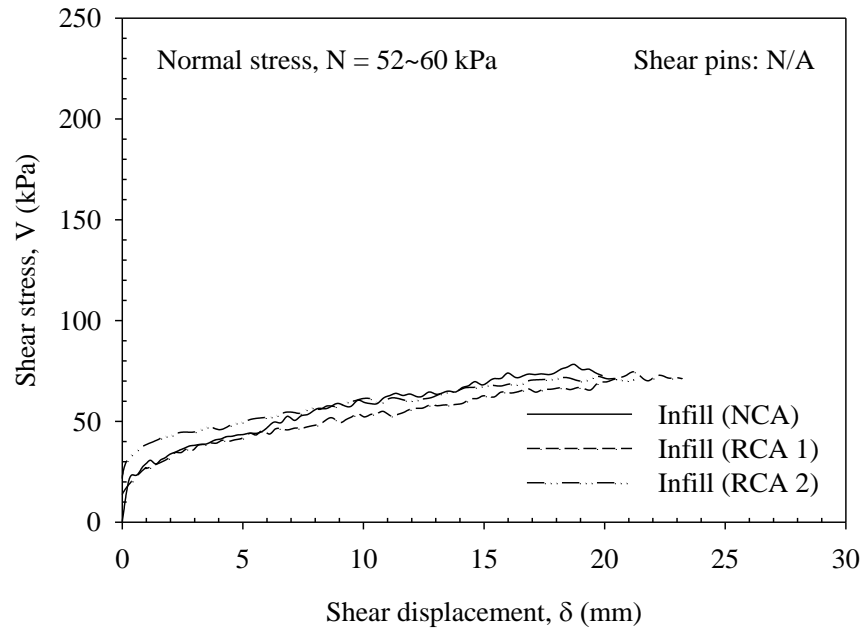


Figure 6.10: Shear stress versus displacement (hollow facing unit with different types of granular in-fills)

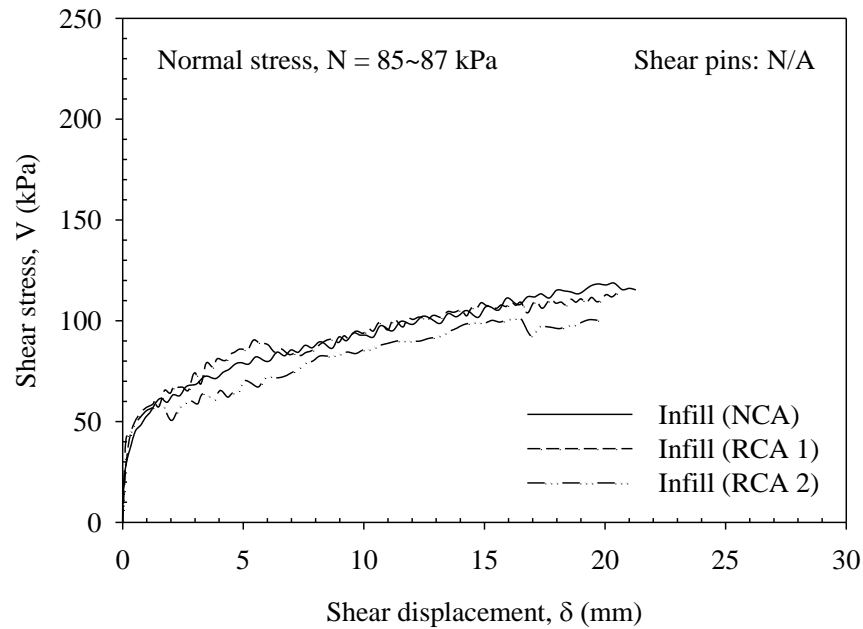


Figure 6.11: Shear stress versus displacement (hollow facing unit with different types of granular in-fills)

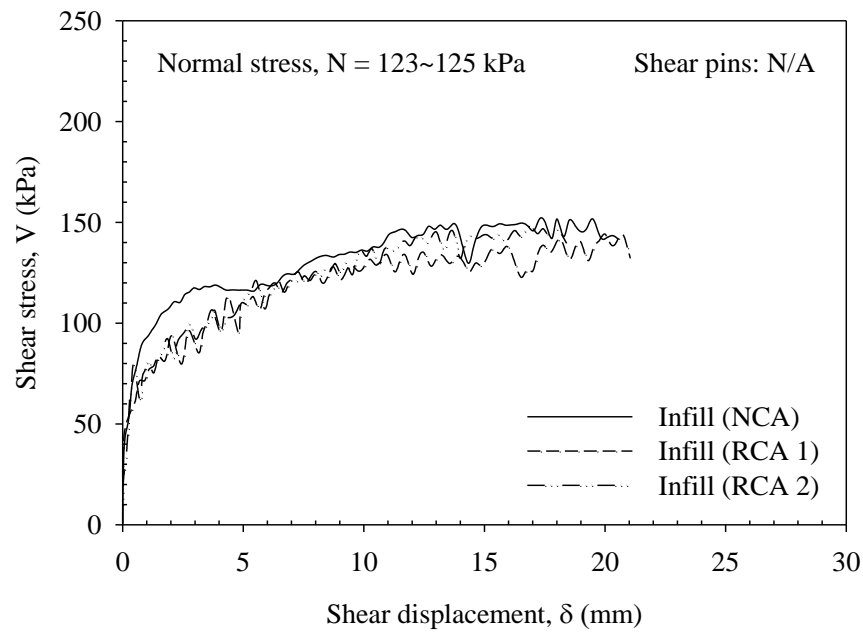


Figure 6.12: Shear stress versus displacement (hollow facing unit with different types of granular in-fill)

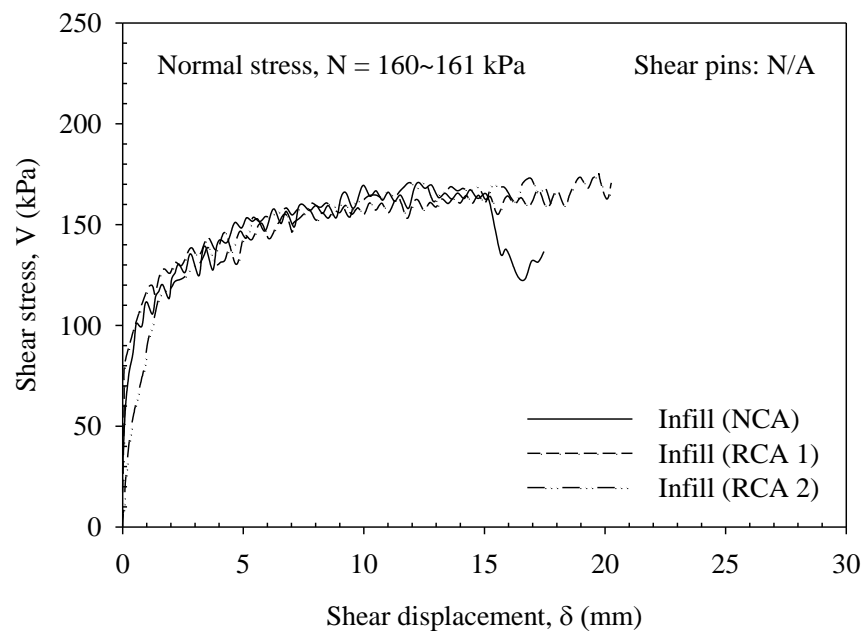


Figure 6.13: Shear stress versus displacement (hollow facing unit with different types of granular in-fill)

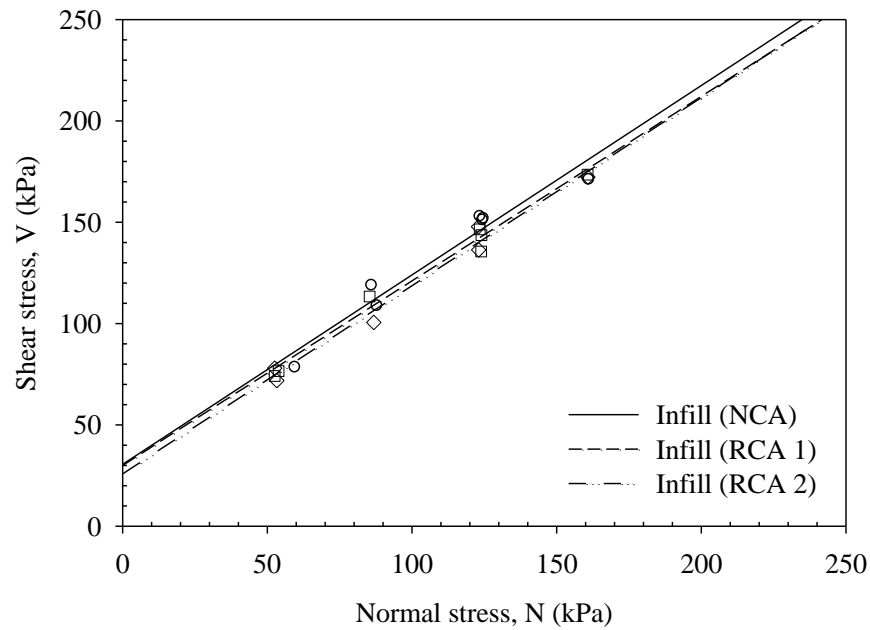


Figure 6.14: Interface shear capacity versus normal stress (hollow facing unit with different types of granular in-fill)

Table 6.1: Interface shear parameters of the tested block system for different types of in-fills

Granular infill	Angle of friction, λ (deg.)	Apparent cohesion, a (kPa)
NCA	43.1	30.5
RCA 1	42.3	30.0
RCA 2	42.9	25.9

6.5 Effect of flexibility of geosynthetic inclusion on the interface shear capacity of hollow infilled segmental concrete units

The influence of flexibility of geosynthetic inclusion on the interface shear capacity was determined by comparing the results of Type 9 (flexible geogrid), Type 12 (stiff geogrid) and Type 15 (flexible geotextile) with Type 8 (no inclusion). Here the data of Type 8 configuration were selected as a referenced data for this configuration because of its interface condition in which hollow blocks infilled with NCA and interlocked with plastic pins.

Test results were presented in the form of shear stress-displacement relationships to compare the effect of different types of polymer reinforcements at interface. Peak shear capacity envelopes were also drawn using Mohr-Coulomb failure criterion to outline the angle of friction for different inclusions.

Figures 6.15 to 6.18 illustrate the typical shear stress-displacement curves of the infilled block system with plastic shear pins for different types of geosynthetic inclusions and different normal stresses. For all series of tests, shear stress increases gradually without any significant rises and falls, and reached the maximum value after a significant amount of shear displacement of about 20 mm. Maximum (peak) shear stresses of the infilled blocks without any geosynthetic inclusion is quite higher than those with geosynthetic inclusions. Due to the presence of geosynthetic inclusions, the maximum shear resistance of the block system with inclusion is lower than that without inclusion. Among the three (3) types of inclusions, polyester geogrid (flexible) performs well than other types of geosynthetics. Even at high normal stresses, the shear strength is very close to no inclusion condition compared to other inclusions explained by the cushion effect of flexible geogrid and its grid structure allowing the aggregate interlocking through the apertures.

The shear stress behavior of the blocks with HDPE geogrid and polyester geotextile inclusion is quite same for all normal stresses. At comparatively low normal stress, shear strength of HDPE geogrid inclusion is higher than PET geotextile inclusion (Figure 6.15). It is also seen that at high normal stress the frictional performance of the blocks with HDPE geogrid inclusion is almost equal to those with polyester geotextile. This is due to the physical characteristics of HDPE geogrid such as thickness of rib and bond, and aperture pattern. The presence of stiff geogrid inclusion at block's interface

reduces concrete-to-concrete frictional contact area and interrupts the aggregates interlocking system partially and hence lowered the interface frictional resistance. At the block's interface, HDPE geogrid works like a friction reducing layer for its stiff and smooth polymeric surface. Its aperture systems also do not give better interlocking mechanism among the aggregates. On the other hand, although, the polyester geotextile provide better cushion at the block's interface but actually it interrupts the aggregates interlocking mechanism fully that caused the reduction of the frictional capacity of the blocks with geotextile inclusion.

The presence of geosynthetic layer at the block's interface reduces the stress concentration that resulted from concrete surface roughness, block alignment and minor variation in block geometry. As a result, it is seen that all the tested blocks remain spalling free at the flanges (Figure A.8).

Plots of the ultimate interface shear stress against the applied normal stress are presented in Figure 6.19. It is seen that, the presence of geosynthetic inclusions reduces the ultimate interface shear capacity of the blocks. Bathurst and Simac (1994), Bathurst and Simac (1997) and Bathurst et al. (2008) observed the same behaviors for different types of block system with geogrid inclusions.

The peak shear capacity envelope of the block system with polyester geogrid inclusion is almost equal to those without any inclusion. This can be explained by the flexibility of inclusion that improves shear transfer across the block's interface than other inclusions.

Figure 6.19 also reports that the reduction in ultimate shear capacity for the inclusions of HDPE geogrid and polyester geotextile is higher than polyester geogrid inclusion. It

may be influenced by the physical structures of the used geosynthetics i.e. flexibility and grid patterns.

Table 6.2 summarizes performance parameters of the infilled block system with and without geosynthetic inclusions. The block system with stiff geogrid inclusion provides lower slope than that with flexible geosynthetic layers and the reduction in friction angle against no-inclusion is about 30.9%. The reductions in friction angle for the presence of flexible geotextile and flexible geogrid are about 15.4% and 2.7% respectively. So, it can also be said that the block system with flexible geogrid performs well and provides better angle of friction than other types of geosynthetic inclusions.

Table 6.2: Interface shear parameters of the infilled block system for different types of inclusions along with plastic pins

Inclusion	Angle of friction, λ (deg.)	Apparent cohesion, α (kPa)
N/A	40.8	52.2
Flexible PET-GG	39.7	46.7
Stiff HDPE-GG	28.2	53.3
Flexible PET-GT	34.5	36.8

Note: N/A = not applicable; GG = geogrid; GT = Geotextile

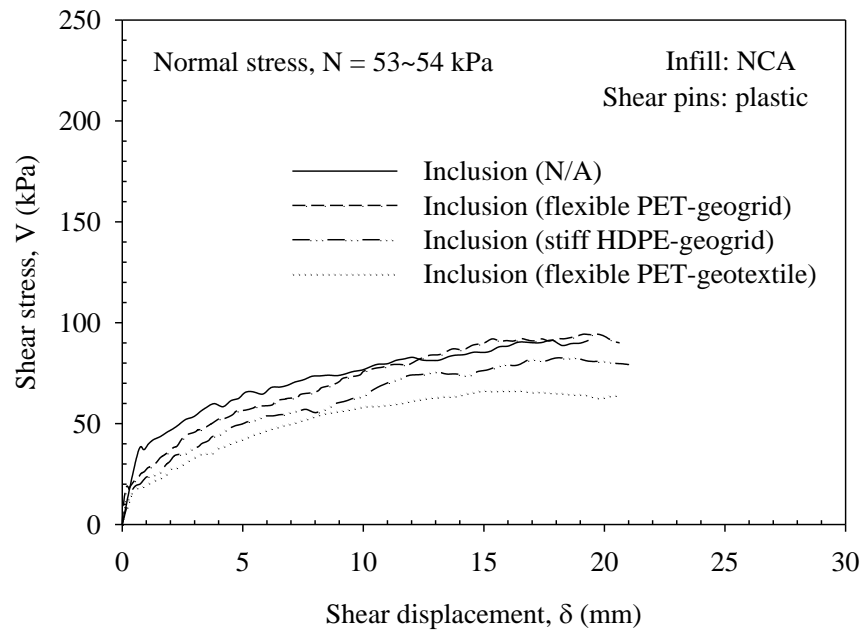


Figure 6.15: Shear stress versus displacement (hollow facing unit with plastic pins, NCA and different types of inclusions)

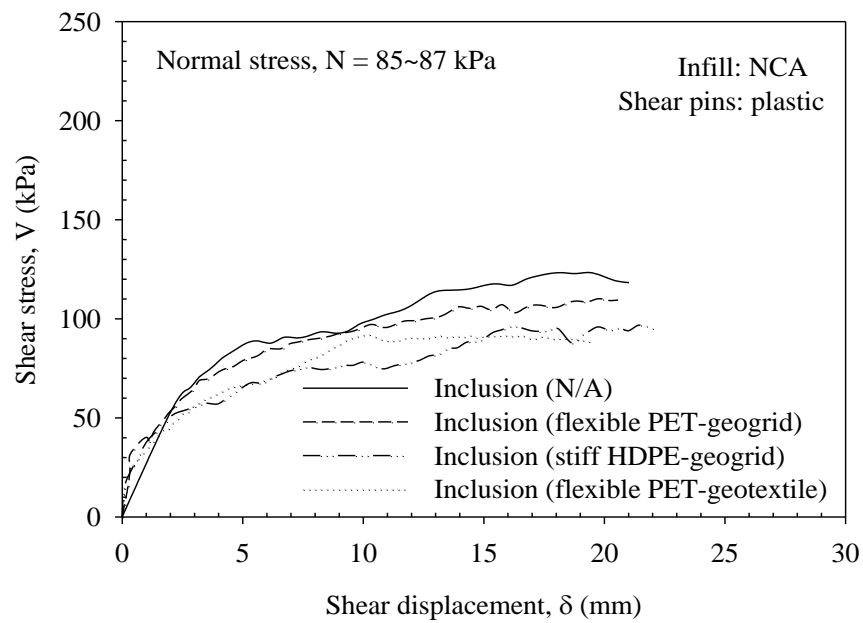


Figure 6.16: Shear stress versus displacement (hollow facing unit with plastic pins, NCA and different types of inclusions)

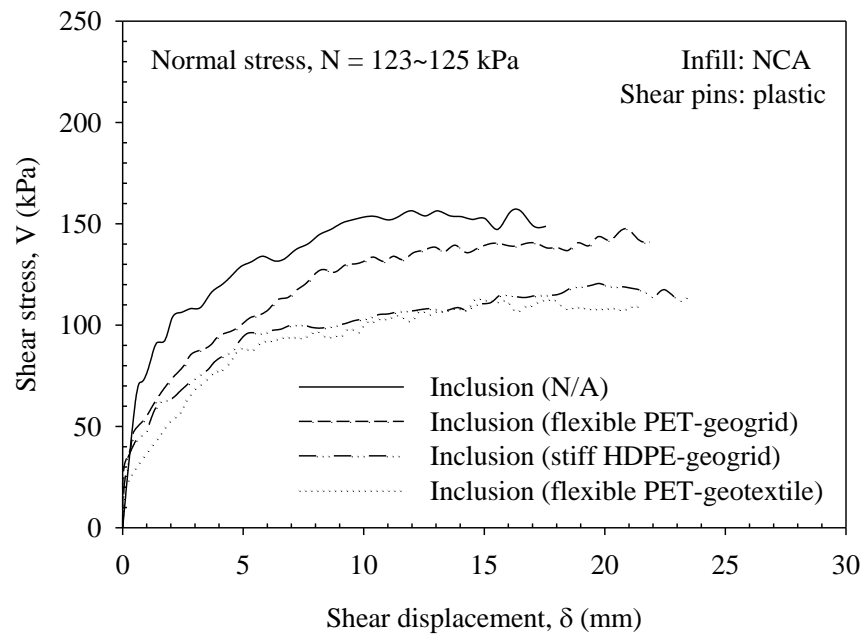


Figure 6.17: Shear stress versus displacement (hollow facing unit with plastic pins, NCA and different types of inclusions)

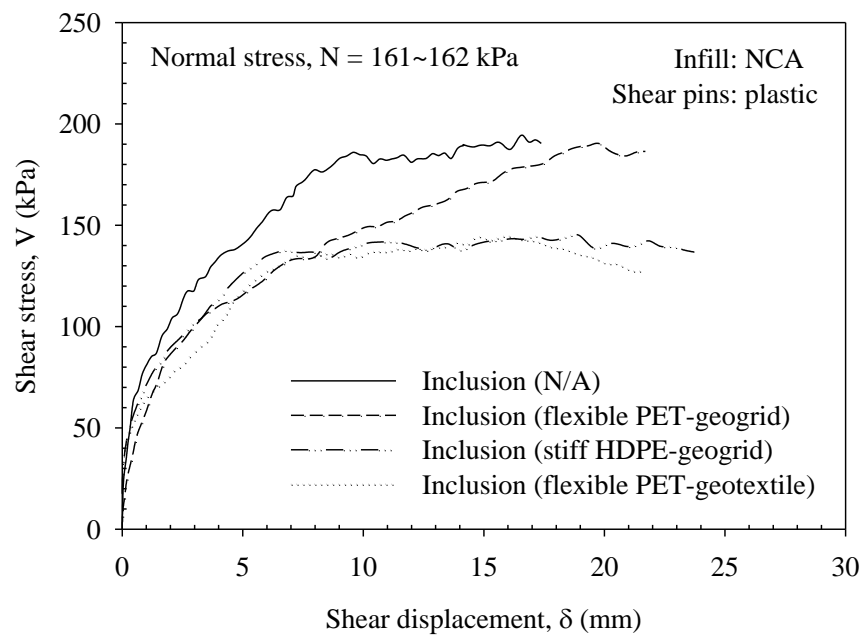


Figure 6.18: Shear stress versus displacement (hollow facing unit with plastic pins, NCA and different types of inclusions)

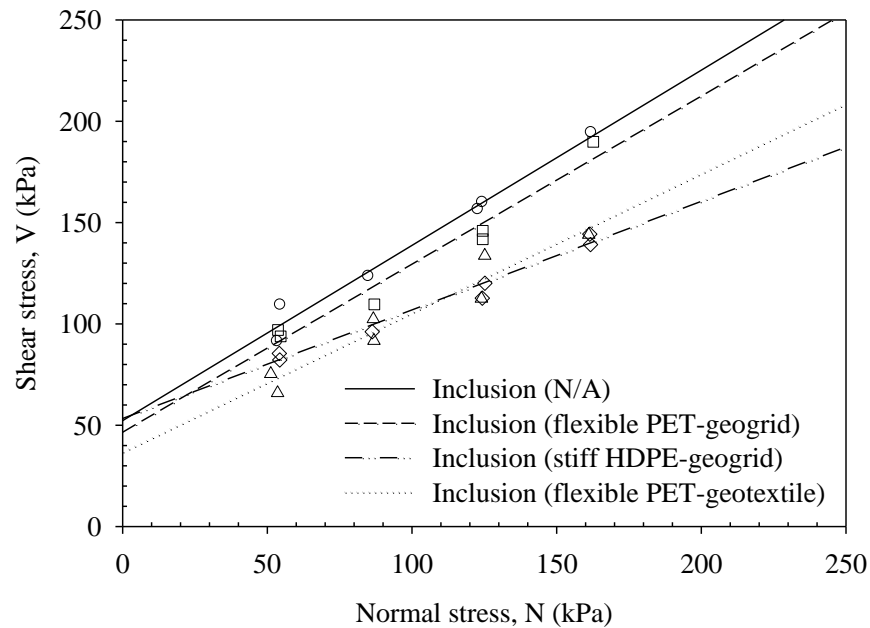


Figure 6.19: Interface shear capacity versus normal stress (hollow facing unit with plastic pins, NCA and different types of inclusions)

6.6 Assessment of shear strength of hollow infilled block system with polymeric inclusions

The frictional performance of hollow blocks with different types of granular in-fills along with geosynthetic inclusions was evaluated by comparing the test results of Types 9 to 17 for each type of polymeric inclusion. The complete laboratory shear tests of the possible reinforced I-Block developed with new type of connection system and recycled aggregates were performed to compare the interface shear capacity for different types of inclusions. The test data were presented in the form of maximum shear against applied normal stress to outline the performance parameters for each case of newly developed reinforced I-Block wall.

Figures 6.20 to 6.22 demonstrate the plots of ultimate (peak) shear capacity of the tested modular block units for different types of granular in-fills and different types of soil reinforcing geosynthetics. It is seen from the Figure 6.20 that shear capacity of the blocks infilled with RCA (#2) for Geogrid 1 inclusion is slightly higher than those NCA. Although, ultimate frictional capacity of blocks infilled with recycled aggregate (RCA 1) is slightly less than those with natural coarse aggregates, which moved away with increment of normal stress. This may be resulted from the angularity and void contents of recycled aggregates, and inter-particles locking mechanism through the flexible geogrid. More or less a similar behavior is observed for test with Geogrid 2 and Geotextile inclusions (Figures 6.21 and 6.22).

Table 6.3 summarizes the performance parameters for I-Blocks under different types of in-fills and inclusions criteria. It is viewed that angles of friction are quite close to each other for the used in-fills with the presence of a certain type of geosynthetic. But the angles of friction for the infilled blocks with different types geosynthetic inclusions are more deviated, which happens due to the physical structures of the used geosynthetics i.e. flexibility and grid patterns. Although, flexible geosynthetics improve shear transfer across the block's surface than stiff geosynthetics but the presence of flexible geotextile interrupts the inter-particle locking fully at the interface plain hence reduced the shear resistance than flexible geogrid. The average (mean) peak friction angle for the blocks infilled with different types of aggregate with flexible geogrid (#1) is about 38.8° and the maximum variation of peak angle of friction from the mean value is about 6.2% for RCA 1. For stiff geogrid (#2), the mean peak angle of friction is about 27.6 and the maximum variation of peak angle of friction from mean the value is about 14.13% for RCA 1. On the other hand, the mean (average) peak angle of friction for flexible geotextile inclusion is about 33.3 and the maximum variation of peak angle of friction

from mean the value is about 5.4% for RCA 2. So, it could be said that flexible geogrid is better as a inclusion than other types of inclusions and the reduction in peak shear capacity is governed by the inclusion's characteristics (flexibility and grid patterns) rather than granular in-fills.

The use of recycled aggregates as granular in-fills in geosynthetic reinforced segmental retaining walls has a concern about the alkalinity of the recycled aggregates ($\text{pH} > 9$), which might influence the shear strength for certain polymeric reinforcements. Alkalinity has potential effect on geosynthetics strength and some of geosynthetic polymers are susceptible in high alkalinity environment (Elias et al., 1998). From the laboratory investigation, it was found that the pH of RCA1 and RCA2 were 8.76 and 11.42 respectively.

The drop of alkalinity of recycled aggregates could be influenced by carbonation of pure concrete and effect of palm oil fuel ash which used as cement replacing materials in concrete to increase its compressive strength. By considering the alkalinity of RCA1 ($\text{pH} < 9$), it may be used for all types geosynthetic reinforced retaining walls. But recycled aggregates with high alkalinity ($\text{pH} > 9$) may be appropriate for those types of geosynthetic reinforcements which are made from polypropylene (PP), polyethylene (PE) and polyamide (PA) polymers (Shukla, 2002). Polyester geosynthetics with polyvinyl chloride (PVC) coating may also be used for recycled aggregates (Pang, 2012).

Table 6.3: Interface shear parameters of block system infilled with different types of in-fills for different types of inclusions

Granular infill type	Geogrid 1		Geogrid 2		Geotextile	
	λ (deg.)	α (kPa)	λ (deg.)	α (kPa)	λ (deg.)	α (kPa)
NCA	39.7	46.7	28.2	53.3	34.5	36.8
RCA 1	36.4	47.8	23.7	48.7	34.0	29.3
RCA 2	40.3	46.1	30.8	47.4	31.5	44.2

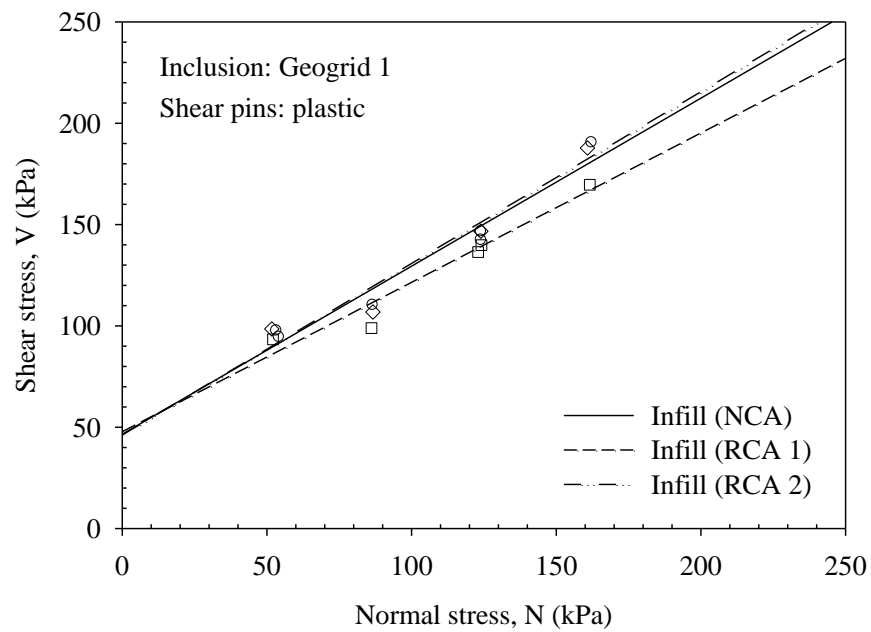


Figure 6.20: Interface shear capacity versus normal stress (hollow facing unit with plastic pins, different types of in-fills and Geogrid 1)

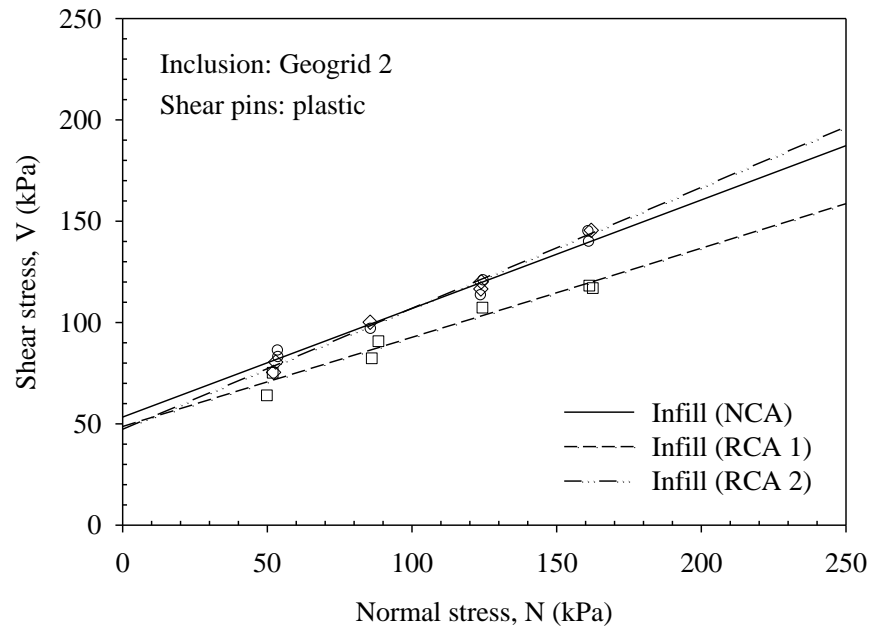


Figure 6.21: Interface shear capacity versus normal stress (hollow facing unit with plastic pins, different types of in-fills and Geogrid 2)

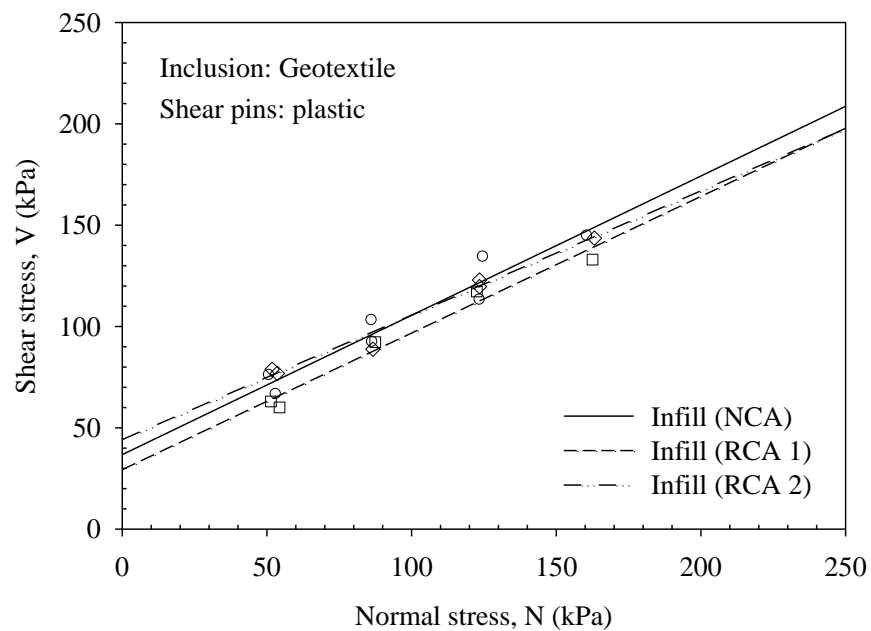


Figure 6.22: Interface shear capacity versus normal stress (hollow facing unit with plastic pins, different types of in-fills and Geotextile)

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

This thesis has shed light on the design and development of a new test facility for full scale laboratory study of I-Blocks (segmental retaining wall units), which is newly designed and fabricated segmental concrete block system in Malaysia. The effects of interface conditions on shear capacity of the innovated I-Block system have been investigated in this research program. A full scale experimental program (laboratory) was conducted. This was conducted the modified apparatus to find out an effective and efficient connection system for I-Blocks and with a view to utilizing the recycled aggregates as granular in-fills, and to identify the effects flexibility of geosynthetic inclusion on the frictional capacity of the newly developed segmental block system. The results of different configurations of interface shear tests have been presented and discussed in Chapters 5 and 6. The analyses and interpretations of the data obtained from all types of test configurations (Types 1 to 17) had reported some important conclusions about the frictional behavior of the segmental concrete block system. These conclusions are included in this chapter in a summarized form. Lastly, recommendations are outlined for future study at University of Malaya.

7.2 Conclusions

The main contributions of this research are the development of test apparatus and modification of connection system for the innovated I-Block produced in Malaysia.

This study also contributes to the sustainable development of segmental retaining wall constructions. Based on the work presented in this thesis the important conclusions are drawn according to the following order:

7.2.1 Performance of the modified test apparatus

After reviewing the NCMA SRWU-1 (1997), NCMA SRWU-2 (1997), ASTM D 6916 (2006) and ASTM D 6638 (2001) test protocols, it was found that protocols recommend a fixed vertical actuator with roller or airbag arrangement. Bathurst et al. (2008) reported that normal loading arrangement greatly influences the performance parameters of different block systems. From the investigation, it was concluded that fixed vertical actuator with flexible airbag arrangement provides better loading arrangement that keeps the normal load constant over the period of shear testing. Fixed vertical piston/actuator without airbag arrangement increases normal load with shear displacement due to bending of vertical actuator locked with the top block during shear loading. By considering the effect of fixed vertical loading arrangement, in this study the test facility was fully redesigned and modified in terms of normal loading arrangement, capacity and gripping systems. In this modified apparatus, a moveable vertical loading assembly was designed to allow the movement of piston attached with top blocks during shear testing. From the investigation it was found that vertical/surcharge load stayed constant over the period of shear testing. So, it can be said that the test facility developed at University of Malaya was successfully modified to impose a constant surcharge/normal pressure over the segmental concrete block systems for full scale laboratory study.

The competitive advantages of the modified apparatus can be summarized as follows:

- Moveable vertical loading assembly provides constant surcharge load with respect to fixed vertical actuator. It is also uncomplicated and time-saving testing arrangement regarding to airbag arrangement recommended by available test protocols.

- It is a well-suited device for full scale laboratory study for all types of facia units (SCUs). The apparatus can be dismantled and adjusted according to the block geometry and test setup.
- The apparatus can easily be used for and applied in full scale laboratory study of relatively high and long wall system because of its capacity and loading assembly.
- A newly designed roller gripping and loading clamp provides better gripping and tensile force distribution along geosynthetic layer.
- The apparatus offers a wide range of displacement speed (1mm/min-60 mm/min) for horizontal actuator.

7.2.2 Rigidity of shear pins and its effect on shear strength

Mechanical shear connectors have great influence on interface shear capacity of facing units although their principle purpose to help out unit alignment and control the wall facing batter (Bathurst and Simac, 1997 and Bathurst et al., 2008). This investigation divulges that the presence of connectors influence the interface shear capacity depending on the nature of the connectors i.e. rigid or flexible. In this study, two types of shear pins (steel and plastic) were used and the effects of the rigidity of those shear pins are summarized as follows:

- Shear pins are one type of mechanical connectors that increase the interface shear capacity of facing units by providing additional interlocking between the layers of those segmental concrete blocks.

- Steel shear pins initially increase the shear strength (initial peak capacity) than purely frictional capacity of empty block system. But due to its rigidity (stiffness) and strength, segmental concrete blocks rupture at the connection joints at a relatively small displacement (<6 mm) and consequently reduce the contact area as well as the interface shear capacity.
- Due to the high stiffness, steel pins do not fail in shear just bend at high shear force and hence increase apparent cohesion (normal stress-independent shear strength). On the other hand, reduction in angle of friction may be influenced the reduction of contact area happened due to steel pins (rigid)
- Flexible connectors provide higher interface shear capacity because no reduction of contact surface area happens during shearing. Plastic shear pins allow the full mobilization of the interface shear capacity of the block system by failing itself in clear shear.
- The segmental block system with or without plastic shear pins easily follow serviceability criterion but the system with steel pins are unable to follow that criterion because these rigid pins breaks the block before serviceability deformation (6 mm for I-Block wall).
- Although, plastic and steel pin were used as shear transferring device in this study to provide additional interlocking for the infilled block system but it is seen that plastic pin provides a better and effective shear connection to the block system in respect to shear strength and serviceability criterion.

7.2.3 Performance of recycled aggregates as granular in-fills

Among different types of segmental concrete units, hollow units are widely used as a facing column for reinforced soil retaining walls because of its cost-effectiveness and other technical facilities like ease of handling. The cavities of the hollow concrete blocks are mainly filled up with granular in-fills to provide better interlocking among the courses of the facing units like mechanical connectors (Selek, 2002; Astarci 2008). Natural (fresh) aggregates is especially used as in-filler in segmental retaining wall construction, which is unsustainable (annihilation of natural resources) and expensive. In this study, two different types of recycled aggregates as granular in-fills were used to compare its frictional performance against fresh aggregate. Based on the frictional behavior of segmental concrete units infilled with different types of granular in-fills, the following conclusions are drawn:

- Granular in-fills increase the interface shear capacity of the hollow block systems and it is much higher than no-infill condition (empty).
- Granular in-fills not only increase the angle of internal friction but also increase the apparent cohesion (normal-stress independent shear strength) of the system. This may happen due to the interlocking mechanism of the crushed gravels, which enhances the positive interlock between the blocks and also increases the self-weight of hollow units.
- Interface shear capacity (peak and service state) of the blocks infilled with the recycled concrete aggregates (RCA) is almost equal to those with natural coarse aggregate (NCA).

- The compressive strength of the source waste concretes has a little or no effect on the frictional performance of recycled concrete aggregates used into facing units.
- The use of recycled aggregate has great advantages on the concrete waste minimization and sustainable developments. So, recycled concrete aggregate (RCA) may be selected as another alternative to infill materials used for segmental retaining walls.

7.2.4 Effect of flexibility of geosynthetic inclusion

Inclusion of a geosynthetic layer at the interface has great influence on interface frictional performance of segmental retaining wall units. It depends on the flexibility of geosynthetic reinforcements as well as block's interlocking system (Bathurst and Simac, 1994; Bathurst and Simac, 1997, Bathurst et al., 2008). By considering the effect of geosynthetic inclusions, three types of geosynthetic reinforcements were chosen and used in the investigation to find out their influences on the interface shear capacity of newly designed and developed precast I-Block system. The following major conclusions are drawn from the comprehensive study about geosynthetic inclusions:

- The presence of geosynthetic layer at the facing unit's (segmental concrete unit) interface reduces the interface shear capacity.
- It depends on the flexibility of the used geosynthetic samples and its grid patterns.
- Flexible geosynthetics improves shear transfer across the block's surface than stiff geosynthetics but the presence of flexible geotextile interrupts the inter-

particle locking fully at the interface plain hence reduced the shear resistance than flexible geogrid.

- The angle of friction of the blocks with polyester geogrid inclusion is higher than those with HDPE geogrid and polyester geotextile inclusions.
- The presence of geosynthetic layers minimizes the localized stress concentrations at the interface as well.
- The block system with stiff geogrid inclusion provides lower slope than that with flexible geosynthetic layers and the reduction in friction angle against no-inclusion is about 30.9%. The reductions in friction angle for the presence of flexible geotextile and flexible geogrid are about 15.4% and 2.7% respectively.
- So, it can also be said that the block system with flexible geosynthetic especially flexible geogrid performs well and provides better angle of friction than other types of geosynthetic inclusions.

7.2.5 Assessment of shear strength between polymeric inclusions and recycled aggregates used as in-fillers in hollow block system

The frictional performance of the recycled aggregates used in hollow block system was investigated with different types of inclusions to evaluate the use of recycled aggregates as alternative granular infill for geosynthetic reinforced segmental retaining wall (GR-SRW). Besides frictional behavior, the use of recycled aggregates as granular in-fills in geosynthetic reinforced regimental retaining walls has a concern about the alkalinity of the recycled aggregates ($\text{pH} > 9$), which might influence the strength for certain polymeric reinforcements. Alkalinity has potential effect on geosynthetics strength and some of geosynthetic polymers are susceptible in high alkalinity environment (Elias et

al., 1998). The major conclusions relating to recycled aggregates with geosynthetic inclusions are summarized as follows:

- The frictional performance of recycled aggregates with geosynthetics in segmental block systems is as good as fresh aggregates used as infill.
- The flexible geogrid is better as an inclusion than other types of inclusions and the reduction in peak shear capacity is governed by the inclusion's characteristics (flexibility and grid patterns) rather than granular in-fills.
- So, recycled concrete aggregates (in terms of frictional performance) may be used as granular infill in segmental retaining wall constructions. But alkalinity ($\text{pH} > 9$) of recycled aggregates may have effects on strength degradation of certain polymers which have low resistance to alkalis e.g. polyester (2 to 9). In such a case, facing stability (shear strength and connection strength) could be influenced by alkalinity of recycled aggregates.
- Polyester geosynthetics with polyvinyl chloride (PVC) coating may also be used for recycled aggregates (Pang, 2012).
- It could be safe, however, to use recycled aggregates of high alkalinity ($\text{pH} > 9$) for the geosynthetic reinforcements which have high resistance to alkalis.

7.3 Recommendations for future study

The scope of the study presented in this thesis has been limited significantly to the two aspects. Firstly it is limited to the design and development a test facility for full scale laboratory study of segmental retaining wall units at University of Malaya. Secondly it deals with the investigation of interface shear testing of the newly designed and locally produced I-Block system. Several potential aspects for further study were identified

during the investigation of this research program. The following recommendations are made to continue the research program:

1. In this investigation, normal and shear force data were collected through the pressure transducers those which were installed at actuators and calibrated against load cell. So, test facility could be further modified by installing load cells at the actuators to acquire precise data of forces directly from load cells.
2. According to the developed test facility, two types of support rails for horizontal actuator were used for different combinations of test setups and the changing of support rails was quite troublesome and time consuming. So it could be better to redesign one support rail for horizontal actuator for all types of test setup and block systems.
3. The investigation of the research has focused on only ultrahigh molecular weight polyethylene (UHMWPE) plastic pins. So there is a scope to study in details about other types of plastic could be used as shear pins and their possible effects on the interface shear strength.
4. Need a more detailed study about the alkalinity effect of recycled aggregates to strength of geosynthetic reinforcements before implication of recycled aggregates as granular in-fills in geosynthetic reinforced segmental retaining walls.
5. This study should be continued to find out the connection strength of the innovated I-Block system with the proposed plastic shear pins. Then a further parametric or numerical study could be performed using the performance parameters of this block system to understand the probable field behavior of the reinforced I-Block walls.

6. The geometry of I-Block could be modified in design because from the laboratory experiment it was found that especially the joint (web to flange) comparatively weaker than other parts of the block. The modification in block's geometry should be done by considering the economical (use of concrete volume) and workability (installation) aspect of the block.
7. Segmental retaining wall units could also be produced from recycled aggregates to make environment more friendly and sustainable.

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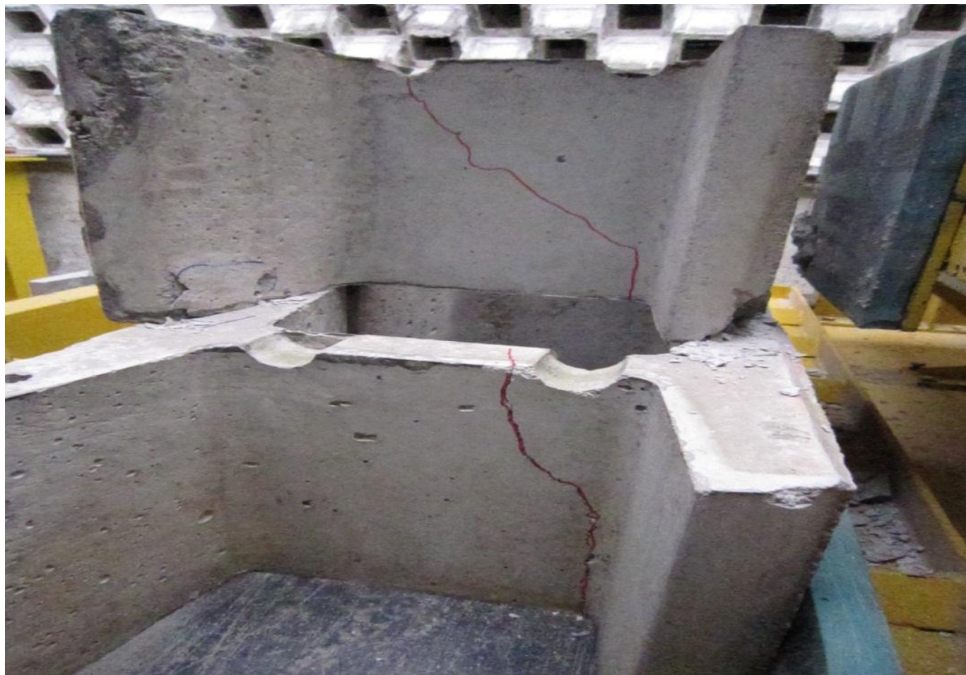
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APPENDIX A: FAILURE PATTERNS FOR DIFFERENT CONFIGURATIONS OF TESTS



(a) Photograph of Type 1 showing corner view



(b) Photograph of Type 1 showing side view

Figure A.1: Failure patterns of empty block at high normal stress of about 160 kPa

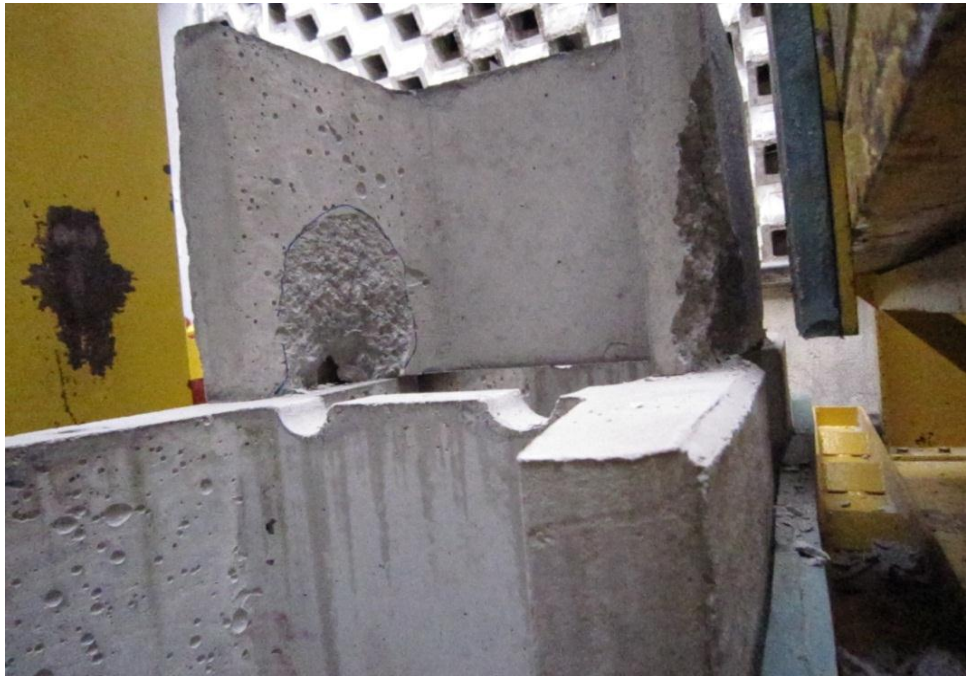


Figure A.2: Photograph of purely frictional shear test showing spalling of top block at connection and rear flange area



Figure A.3: Photograph of plastic shear pins showing failure patterns (clear shear and bending)



Figure A.4: Photograph of steel shear pins showing failure patterns (bending)



(a) Spalling and cracks at rear and front flange respectively

Figure A.5: Photograph of common failure patterns of empty block system with steel shear pins



(b) Triangular crack at joints of top block



(c) Spalling at the joints of bottom blocks

Figure A.5 (continued): Photograph of common failure patterns of empty block system with steel shear pins



(d) Straight and triangular cracks at the bottom blocks propagated from the joints



(e) Complete straight crack through connection joint

Figure A.5 (continued): Photograph of common failure patterns of empty block system with steel shear pins



Figure A.6: Photograph of the infilled block system with plastic shear pins showing shear failure of shear pins



(a) Spalling at connection joint and rear flange of top block

Figure A.7: Photograph of common failure patterns of the infilled block system with steel shear pins



(b) Spalling at connection joints of the infilled bottom layer with bended steel pins

Figure A.7 (continued): Photograph of common failure patterns of the infilled block system with steel shear pins

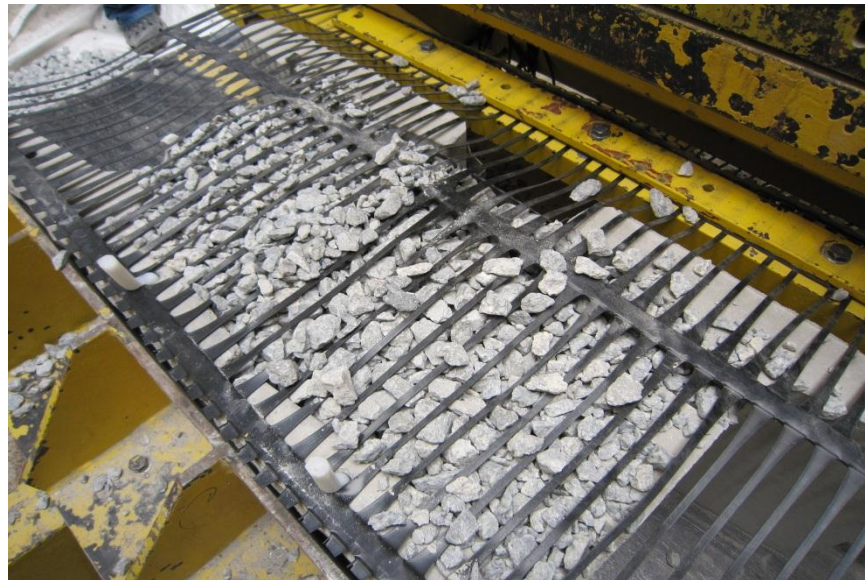


(a) Shear failure of plastic pins without any significant spalling or cracking in blocks

Figure A.8: Photograph of common failure patterns of the infilled block system with inclusion



(b) Rubbing of flexible (PET) geogrid layer without any rupture



(c) Stiff (HDPE) geogrid inclusion at the interface

Figure A.8 (continued): Photograph of common failure patterns of the infilled block system with inclusion



(d) Flexible (PET) geotextile inclusion at the interface

Figure A.8 (continued): Photograph of common failure patterns of the infilled block system with inclusion